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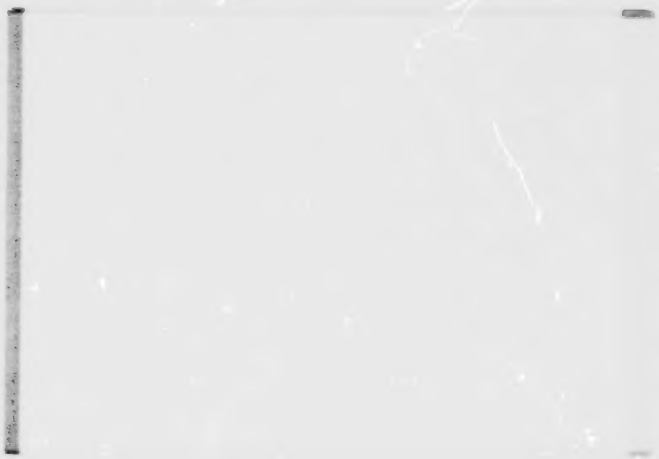

DEVELOPMENT OF METHODOLOGY FOR THE ANALYSIS OF RESIDENTIAL FLOOR TRUSSES

Submitted to:

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1. INTRODUCTION

1.1 Contract and Sponsorship

This report has been prepared as part of the work undertaken by Morrison Hershfield Limited (MH) for Supply and Services Canada (DSS) under contract number 43SS.1P001-3-5610-9. The contract was awarded as a result of an Unsolicited Proposal No. UP-M-375 dated July 22, 1982. It was executed by DSS on behalf of Canada Mortgage and Housing Corporation (CMHC), the sponsoring department, and was managed by the Science Procurement Managers, Mr. E.C. Carey and Mr. Ralph Finlayson of DSS.

1.2 Acknowledgements

The scientific and technological content of the work performed under this contract has been subject to periodic examination and approval by the Scientific Authority. Originally, this was Mr. Anthony Karwoski of CMHC; later, it was Mr. Fred Plimmer of CMHC. He was assisted by a Floor Truss Study Advisory Committee which comprised the following members:

Mr. Dan Anderson - CMHC, Ottawa (Chairman)
Mr. Robert Hopkins - R.J. Hopkins and Assoc. Ltd., Keswick
Mr. Eric Jones - Canadian Wood Council, Ottawa
Mr. Glen McLachlan - CMHC, Ottawa
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1.3 Abbreviations

The following abbreviations have been used in this report:

CMHC - Canada Mortgage and Housing Corporation
CSA - Canadian Standards Association
CWTI - Canadian Wood Truss Institute
DSS - Supply and Services Canada
MH - Morrison Hershfield Limited
S-P-F - Spruce-Pine-Fir (lumber species group)

2. BACKGROUND AND OBJECTIVES

2.1 Background

A recent development in the engineered component approach to floor system construction is the metal web connected floor truss which has evolved as an outgrowth of the truss plate industry. In the early years of residential roof truss construction, the truss members were fastened together with nailed plywood gusset plates. The plywood has now been virtually eliminated from truss construction by truss plates which are manufactured of light gauge metal and which have integral teeth stamped from the metal. These plate type connections are either pressed or rolled into the wood, thus obviating the need for addition of nails and speeding up the assembly process.

Although the original plate connection was intended primarily for pitched roof trusses, it soon found its way into deep parallel chord roof trusses and then into relatively shallow parallel chord trusses intended for floors. Today these parallel chord trusses constructed of lumber chords and lumber webs with truss plate connections are being installed in new residential construction and are apparently performing satisfactorily. However, the number of short lumber webs which must be cut and handled during truss assembly makes for a somewhat labour-intensive production process.

To overcome the production problems associated with short lumber webs, some truss plate manufacturers have come up with an innovation - the metal web. The web is preformed in metal gauges equal to those used for truss plates and each web has integral teeth stamped into its ends so that it may be pressed, as a unit, into the sides of the chords. At the present time, there are three major types of floor trusses marketed in Canada: a wood web truss and two types of metal web trusses - one uses vee's and the other has single diagonal members ("tension struts") combined with wood verticals. All three types of floor trusses are represented in the present project.

The wood floor truss has proven its viability in the market place and it is assured of taking an increasing share of the market for floor structures.

We do not have a suitable method for designing metal web trusses. We do not adequately know how to determine in advance whether metal web connected floor trusses will perform satisfactorily and provide an adequate factor of safety against collapse. In particular, this situation has been a problem for CMHC because this agency must decide whether a given design of floor truss may be considered acceptable for use in new homes financed under the National Housing Act.

Up to the present time CMHC has taken the prudent approach of requiring that floor assemblies of this type be tested in full size and that the test units meet specific serviceability and safety criteria. This method calls for testing of maximum spans under uniform load where elastic deflections are likely to govern design, but no testing is required for short heavily-loaded spans where the strength of the metal web may govern the design. Another limitation of the current procedure is that it does not provide the information that a designer needs to account for point loads, cantilevers, stress reversals or chases. In short, full scale testing is expensive, time consuming, repetitious and does not really provide enough information to carry out satisfactory analyses of floor truss behaviour or to optimize their design.

It is interesting to compare the present state-of-the-art for floor trusses with the earlier evolution of plate connected roof trusses. These latter components went through a similar period of full scale testing and in 1971 the industry set up an association (The Truss Plate Institute of Canada) and engaged MH to draft a standard for the testing of truss plate connectors. In 1976 the draft standard became a CSA Standard (S347) and today, for every truss plate used in Canada the design values are determined by this test method. The design values developed through the plate tests are combined with lumber design values so that standard engineering practices may

be used to size connections, webs and chords for any given truss application. This meant that the full scale test approach which had evolved into CSA Standard S307 in 1977 is no longer required. The plate tests, in combination with new information on the strength of lumber, provided a more satisfactory and less expensive basis for design.

On June 24th, 1982 a Symposium on Wood Floor Trusses was organized by Mr. G. Walt of CMHC. At this symposium the state-of-the-art in wood floor trusses was reviewed and, among other points, it was confirmed that full scale testing of floor trusses is an expensive option that in the long run does not provide the best answers for design purposes. The Symposium participants recommended that research be conducted to develop inexpensive methods of measuring the static properties of metal web trusses and to develop a mathematical model which will predict, on the basis of small specimen tests, the behaviour of full scale metal web trusses under various loading conditions. It was also noted at this Symposium that the vibration performance of floor trusses can be expressed as a function of the static stiffness of the floor elements and, therefore, that improved information about the static properties of floor trusses was needed.

2.2 Objectives

In response to the above background, the present study was developed under the following objectives:

a) to develop a standard method of test for metal webs used in floor trusses,
b) to develop a method of structural analysis which will predict deflections and strength of floor trusses on the basis of the properties of the chord members and of the web members,
c) to test the efficacy of this proposed structural analysis method by comparing performance predicted by it with the performance of full scale floor trusses under test, and
d) to study the behaviour of full size floor trusses subject to long term loads.

3. FULL-SIZE SHORT-TERM TRUSS TESTS

3.1 Experimental Design

The first phase of the experimental work consisted of short-term load tests on eighty full-size floor trusses.

As indicated earlier, the three major types of residential floor trusses marketed in Canada are included in this project. All three types have lumber top and bottom chords but the webs are different; they are:

(i) metal "vee" webs (type MW),
(ii) metal "tension strut" diagonals with wood verticals (type TS), and
(iii) wood webs (type WW).

For each truss type, four different designs were chosen from the truss manufacturers' catalogues. Two depths typical of each manufacturer's product were selected. The floor loading was set at:

- top chord: live load = 1.9 kN/m^2 (40 psf)
  dead load = 0.5 kN/m^2 (10 psf)
- bottom chord: dead load = 0.2 kN/m^2 (5 psf)

Allowing truss spacing to vary, four designs were then selected, and the test span used was the maximum allowable span for that design. To fit into the laboratory test apparatus, no spans over 6400mm were chosen.

All of the truss designs were produced according to the methods recommended by the Canadian Wood Truss Institute (CWTI) in the 1981 edition of its "Truss Design Procedures and Specifications for Light Metal Plate Connected Wood Trusses". The analysis assumes that trusses are comprised of linear elastic pin-connected elements and that chord members are continuous over the panel point supports.

Figure 1 presents a summary of the depths, spans, design loadings and general truss configurations for the four designs of each truss type. Also indicated are those designs which were selected for design modifications. Figure 1 also includes the average test results for each design.

Five replications were tested for each of the original twelve designs. Later, four of the designs were modified and five replications were tested for each modification, for a total of (12 + 4) x 5 = 80 trusses tested.

3.2 Materials

Details of each of the original twelve designs and the four modified designs are in the upper portions of Figures 2 to 17. The top and bottom chords consist of 38 x 89mm No. 2 and better grade S-P-F lumber used on the flat. The web types and arrangements can be seen in the above Figures, and further details of the webs appear in Figures 18 to 22. The web types corresponding to each truss type are shown in Table 1.

The flatwise flexural modulus of elasticity was measured for each piece of lumber to be used in the chords.

The trusses were manufactured in commercial fabricating shops according to standard manufacturing practice. Photos 1 to 4 show the production sequence of an MW truss, and Photos 5 to 8 do the same for a TS truss.

Throughout the fabrication and handling processes, MH requested that no special treatment be given to the test trusses; they were to be made and handled by the manufacturer's standard or routine procedures. In the opinion of MH, the fabrication and handling of the test trusses were generally in accordance with the truss designs and with standard industry practice.

3.3 Preparation of Testing Equipment

The floor truss tests were carried out by MH personnel at their Mississauga laboratory. The testing frame provided a free moving suspended support at one end of the truss and a fixed support for the other end. All trusses were supported at the bottom chord level.

All loads were applied to the top chord using a system of pneumatic jacks. The jacks were placed at 610mm intervals along the length of the truss and pressure was exerted onto the truss through load spreader bars spaced at approximately 305mm intervals. The jacks were actuated by a pump and the resulting support reaction at the free end was measured by a load cell.

The deflection of the truss during load application was measured by a set of rulers graduated in millimetres and a double string line anchored on the test truss at its supports. This method permitted readings to an accuracy of 1mm.

3.4 Test Procedure

Prior to the actual load testing of the trusses, certain observations and preparations were made.

The first step was the collection of detailed observations of truss geometry. In this step, accurate measurements of the following were made and recorded:

- overall truss dimensions
- panel point locations
- plate and web sizes and locations
- gap widths between members and webs at the connections.

In addition, each truss was examined for unusual defects such as long splits, large knots or webs misaligned during the fabrication process. After taking the initial measurements, the trusses were manually placed into the test frame.

The trusses were prepared for deflection measurements by nailing rulers to the bottom chord at midspan of the truss. A double nylon line was then installed near mid-depth of the bottom chord. The line was fixed to a bent metal angle located at the truss support.

With the truss set in position and load/deflection equipment prepared, zero readings were taken on the deflection string line and on the load cells. The load was then applied in the following sequence.

a) The dead load was applied in two load steps.
b) The loads were then increased at equal intervals to a value equal to twice the live load plus the dead load and maintained for a three hour period.
c) The live loads were removed and the dead load was maintained for ten minutes.
d) The load was increased to two times live load plus dead load, then increased in steps to failure.

Deflection readings were taken from the string line and rulers at regular intervals throughout the entire loading cycle of the truss. Zero readings were taken before any load was applied to the truss and then deflections were measured at each load step. At the three-hour hold, deflection reading were taken as soon as the load level was reached, at 15 minute intervals for the first hour and at 30 minute intervals for the remaining two hours. Deflections were measured to the nearest millimetre.

For each truss, a record of load and deflection data was maintained up to the time of failure. The time at which observations were made was noted.

After failure of each truss, a detailed set of observations at the failure location was made. The observations included the load level at failure, photographs of the failure and any relevant information which would help to explain why the truss failed in that manner.

3.5 Results

The results of the original sixty floor truss tests are presented in Figures 2 to 13. The diagrams at the top of the tables illustrate the truss configuration, and show the average panel lengths and truss depths. For each of the five replications of the design, the failure load, the deflection at the design load level and the mode of

failure are also shown. The average value of failure load and of deflection at design load are calculated and transferred to the two right hand columns of Figure 1; deflection at design load is given as a fraction of the span and the failure load is given as a multiple of the design load.

Initially, all of the metal web trusses were assembled with the inner end verticals inserted with their wide face parallel to the span of the truss. Although this was in accordance with the manufacturer's standard practice, it was not the detail specified on the drawings. The first three trusses tested (MWB-1, MWB-2 and MWB-3) were loaded to failure with this detail intact. At failure, it was noted that the top chord of the MWB-2 and 3 trusses had split transversely. Since this splitting was probably a result of a manufacturing anomaly rather than a design consideration, it was decided to correct the detail by adding blocks. In all of the remaining metal web trusses, two 25mm by 38mm blocks were installed between the top and bottom chords adjacent to the outer end verticals.

For the MWA series of trusses (Figure 2), one truss (MWA-4) was seen to have one of its webs underpressed at joint 3 and was removed from the experiment. For the remaining four specimens the dominant failure mode was pullout of the teeth of the vee web from the chords of the truss. This was seen to occur on both the top and bottom chords, but it appears that the failures initiated at the top chords at joint 3. A typical example of this is shown in Photo 9. This failure mode was often accompanied by, or was followed by, other forms of failure such as the tearing away of some wood from the top chord in the vicinity of the teeth; this was seen in the case of truss MWA-3. In the case of truss MWA-5 a secondary form of failure was the compression buckling of the metal leg of the vee as shown in Photo 10. In the case of truss MWA-2 a different mode of failure was seen; this was pullout of the web teeth in tension from joint 22; this is illustrated in Photo 11. Three of the four failures involved the compression diagonal 2-3; therefore, in a modification of this design described later, it was decided to double up this compression diagonal to see if an increase in strength would result.

In the MWB series of trusses (Figure 3), all failures involved the first tension diagonal, either member 1-2 or 13-14. For this design the end tension diagonal was a double member; failure usually took place in withdrawal of the teeth of one of the two pieces of this tension diagonal. This is illustrated in Photo 12. As indicated above, trusses MWB-2 and MWB-3 failed as a result of transverse splitting of the the top chord. Eliminating these two trusses from the results increases the average ultimate load by 2 percent.

For design MWC (Figure 4), two major failure modes were observed. Trusses MWC-2 and MWC-5 had relatively low failure loads (2.77 kN/m for each); in these two cases the failure mode was web tooth pullout in tension at the first or second top chord joint. For the other three trusses the failure mode was buckling of the first compression web (either 2-3 or 12-13), accompanied by tooth pullout at the top chord (Photo 13).

For design MWD (Figure 5), two of the trusses failed by a combination of tooth withdrawal and wood pullout at the top end of the second tension diagonal (at joint 3) as shown in Photo 14. A variety of other failure modes were associated with the other three trusses.

For the tension strut trusses in Figures 6 to 9, the symbols on the web members represent the following: the letter D means the tension struts are on both the front and the back of the truss, the tension strut drawn in solid lines without the letter D means that the strut is on the front side of the truss only, and the tension strut drawn in broken lines means that the strut is on the back side of the truss only.

For design TSA (Figure 6), all five trusses failed as a result of tooth widthdrawal in the bottom chord splice, as shown in Photo 15. The obvious modification to this design, as described later, was to increase the size of the tension splice plate.

For design TSB (Figure 7), four of the five failures occurred as a result of the teeth in the end of the first tension diagonal pulling out of either the top chord or the bottom chord, as shown in Photo 16. The other failure, which occurred in truss TSB-5, was caused by a tension failure in the metal itself at the top end of the first tension diagonal, as shown in Photo 17.

For design TSC (Figure 8), all of the trusses failed as a result of the teeth at the ends of the first tension diagonal pulling out of the chords. In some cases this was accompanied by pulling out the wood fibre and by tension failure in the metal itself at the gripping area.

For truss design TSD (Figure 9), the end metal tension strut was replaced by two lumber tension diagonals joined to the chords and to the vertical by means of truss plates. In four of the five trusses the connection of this truss plate to the bottom chord proved to be the weak link in the design and failures occurred at that point. Again, this suggested that the design could be modified by changing from the wooden diagonal members to metal tension struts, as discussed later. The tooth withdrawal failure mode is shown in Photo 18.

For the WWA trusses (Figure 10), all failures were in the bottom chord. In four of the five trusses the failure was in the bottom chord splice and in the remaining case it was a tension failure of the bottom chord itself at midspan. The bottom chord splices failed by tension in the metal as seen in Photo 19. The bottom chord tension failure in truss WWA-1 occurred at a large knot (Photo 20).

For the WWB trusses (Figure 11), a new kind of failure mode was observed in three of the five trusses. This was a failure in the bottom chord due to a combination of chord bending and axial tension at the plate locations; it is illustrated in Photo 21. The other two failures were bending failures in the chords at locations of large knots.

For design WWC (Figure 12), the failure pattern was identical to the WWA series, i.e. four trusses failed at the bottom chord tension splice and one failed (at a lower load) in the bottom chord at mid-span at a large knot.

Likewise, for the WWD trusses (Figure 13), there were three tension failures in the bottom chord splice and two tension failures in the bottom chord itself near mid-span.

3.6 Modified Designs

After studying the results from the first sixty trusses (twelve designs, five replications), it was decided to modify four of the designs in an attempt to obtain more information about the behaviour of those trusses. The four modifications were as follows:

a) Design MWA (Figure 14) - as indicated earlier three of the four failures in the original series occurred at the top end of the first compression diagonal. Therefore, the design was modified by doubling up the first compression diagonal, as shown in Figure 14. Figure 1 lists the improvements that resulted from this modification: the ratio of failure load to design load increased from 2.02 to 3.31 and the trusses became stiffer. The deflection at design load decreased from L/284 to L/354. This improvement in performance accompanied a change in the failure mode from the compression diagonal tooth withdrawal failures to failures in the first tension diagonal at the upper gripping area and also a tension failure in the bottom chord splice.

b) Design MWD (Figure 15) - three of the failures in the original series occurred at the first top chord joint. It was therefore decided to change the arrangement of the metal vees in the first and second panels as can be seen by comparing Figure 5 with Figure 15. This modification, however, brought about essentially no change in the strength of the trusses and only a small increase in the stiffness.

c) Design TSA (Figure 16) - the original trusses all failed by tooth withdrawal in the bottom chord splice. Therefore the splice was enlarged and this modification brought about a change in the strength ratio from 2.37 to 2.63 and increased the stiffness slightly from a design load deflection of L/260 to L/308. The failure mode shifted from the splice plate pullout failure to a pullout failure at the bottom end of the first tension diagonal.

d) Design TSD (Figure 17) - in the original trusses, failure occurred in the truss plates at the bottom end of the first tension diagonal which comprised two lumber members. This diagonal was replaced by metal tension struts on both the front and back of the truss. The failure mode shifted to a pullout failure at the top of the second tension diagonal from the top chord. This was accompanied by tearing away of wood fibre from the top chord. The change in failure mode was likely caused by a softening of the first truss panel when the wood webs were replaced by metal webs. In all of the modified trusses, the end webs had begun to peel off of the chords prior to failure although the ultimate failure occurred at the top of the second tension diagonal. Because the ends of the modified trusses were not as stiff, it appeared that more rotation occurred around the top chord splice. The bending around the splice appeared to produce the tooth withdrawal failure in combination with the tearing of the wood fibre seen in the modified design. This modification resulted in a slight degradation of the truss, i.e. the deflection at design load increased from L/455 to L/431, and the strength ratio decreased from 2.99 to 2.41.

4. SHEAR PANEL TESTS

4.1 Experimental Design

One can think of floor trusses as consisting of two types of elements: chords (top and bottom) and web members. It was evident from the short-term tests on full-size trusses described in the previous chapter that the properties of the webs influenced greatly the strength and stiffness characteristics of the entire truss. In fact, with the exception of those cases where the truss failure mode involved the easily-remedied weaknesses in either the bottom chord lumber or a bottom chord splice, all of the failure modes involved the capacity of the web member and its connection to the chords. It was therefore decided to see if tests on web panels could be used, along with a knowledge of the chord properties, to predict the behaviour of the entire truss. The main activity of the webs is to resist shear forces and therefore a specimen was developed to simulate the shear behaviour of truss web systems.

These test specimens are shown in Figures 18 to 22 and in Photos 22 to 38. A test specimen consists of two web units placed back to back. Each specimen comprises the web element and a portion of the chord members connected to it. Load is then applied to the two chord pieces which are side by side in the middle of the specimen, and the specimen is supported at the bottoms of the outer chord pieces (Photo 22). Lateral supports are also provided to the top and bottom chord pieces so that the specimen deforms in the vertical plane, ie in shear.

Table 1 shows the relationship between the type of shear panel specimen and the type of truss in which it occurs. This table also shows the number of replications that were tested for each type of shear panel. A total of 233 shear panels were tested.

4.2 Materials

The chord material was No. 2 and better grade S-P-F lumber of 38 x 89 mm cross-section. All of the lumber used in the shear panel tests had specific gravities which fell in the range of 0.31 to 0.36. This conforms to the specific gravity

requirements for S-P-F lumber used in the testing of truss plates. The webs were the same as those used in the full-size trusses and the fabrication of the shear panel specimens was done by MH personnel in the same shop that fabricated the wooden web trusses.

4.3 Test Procedure

The vertical compressive load was applied to the double chords in the middle of the composite shear panel. Deformations of the test specimen were measured by mounting a transducer on the lower part of the central double chord and measuring the movement of this point with respect to the fixed point, as shown in Photo 22.

Load was applied continuously to failure with corresponding deformation readings emanating from the transducer.

4.4 Results and Analysis

For each shear panel specimen, the maximum load was recorded and the stiffness of the specimen was determined as the slope of the load versus deformation plot. For each test specimen the results of ultimate load and initial stiffness are given in Tables 2 to 20, and the results are summarized for each shear panel type in Table 1. Also given in Tables 2 to 20 are the failure modes for each specimen.

Observations as to failure mode for each series of shear panels are as follows:

A - is the single tension strut used in the shallower MW trusses. All ten failures took place at joint 2 by either the wood splitting under the plate or the teeth withdrawing from the chord at that point. In two cases some of the wood of the chord was torn away as the teeth withdrew. Photo 23 shows a typical failure.

B - is the MW single compression strut. For seven of the nine specimens, failure was by withdrawal of the truss plate teeth from the chord at either the upper or lower end of the strut. In four cases some pieces of wood were torn away with the teeth, and in four of the cases one or both of the webs also buckled. The remaining

two failures were by splitting of the wood under the plate at the lower end of the strut. Photo 24 shows a typical tooth withdrawal failure accompanied by buckling of one web.

C - is the simple vee used in truss type MW. Within the twenty specimens the most prevalent form of failure was web buckling; in six of the cases the web buckling occurred alone; in the other eleven cases it occurred in combination with other failure modes. The second most prevalent failure mode was withdrawal of the teeth at joint 3, i.e. at the lower end of the compression leg of the vee. Photo 25 shows an example of a combination of tooth withdrawal at joint 3 plus buckling of one of the compression webs.

C(modified) - In these additional ten tests the outside chord members were cut 51 mm longer so that there would be no interference between the bottom of the compression legs of the vee's and the base of the testing machine. As can be seen in Table 1, this modification resulted in an increase in both the ultimate load and the initial stiffness. In this case the failure mode is predominantly withdrawal of the teeth at joint 3 i.e. at the bottom end of the compression leg of the vee. In contrast to the previous series, only two of the ten specimens showed any web buckling.

D - is the reinforced vee consisting of a simple vee with a tension strut and the compression strut added to each side as shown in Figure 18. The failure mode for seven of these ten specimens was splitting of the wood under the plate at the bottom ends of the compression legs of the vee's. This type of failure is shown in Photo 26.

E - is the single tension strut used with the deeper MW trusses. In seven of the ten tests failure was by withdrawal of the teeth from the chord lumber at either the top end or the bottom end. The other three failure modes were by splitting of the chord lumber under the plate. See Photo 27.

F - is the single compression strut used with truss type MW of the 400 mm depth and is comparable to specimen type B for the shallower trusses. Buckling of this

compression strut took place in nine of the ten specimens and was usually accompanied by either tooth withdrawal at the top end of the strut or splitting of the wood under the plate. See Photo 28.

G - is the simple vee configuration used for the 400 mm deep trusses of type MW, and corresponds to specimen type C for the shallower trusses. Almost all of the twenty specimens failed by a combination of tooth withdrawal from the chords and buckling of the web, as shown in Photo 29.

Specimen Type G was modified by cutting off 102 mm from the bottoms of all of the chord members. Ten specimens of the modified Type G panels were tested and, as shown in Table 1, the strength increased on the average from 19.9 to 21.4 kN and the initial stiffness increased markedly from 2400 to 3500 N/mm. There was also a distinct change in the failure mode with only three of the ten specimens showing any web buckling; the dominant failure mode for the modified specimens was tooth withdrawal at either joint 1 or joint 3.

H - is the reinforced vee for the deep metal web trusses corresponding to specimen type D for the shallower trusses. Ten specimens were tested. In almost all cases the specimen failed by splitting of the wood under the plates at joints 6 and 7 as shown in Photo 30.

Specimen H was modified by making all of the chord legs 13 mm longer at the bottom. Four specimens of the modified design were tested. The initial stiffness was virtually unchanged but the ultimate load increased significantly from 23.0 to 28.2 kN. As seen in Photo 31 the failure mode changed from splitting under the plate at joint 6 and 7 to tooth withdrawal at joint 7 accompanied in three cases by web buckling.

I - is the double tension strut used for the shallow metal tension strut (TS) series of trusses. Thirty replications of this type of shear panel were tested and in the great majority of cases the failure was by tooth withdrawal or splitting of the wood under the plate at joint 1. In four cases failure was by tooth withdrawal at joint 3.

J - is the single tension strut used in the shallow TS trusses. Fifteen of these were tested and most failed by tooth withdrawal at either joint 1 or joint 3. In two of the cases the wood split under the plate at joint 1. See Photo 33.

K - is the double tension strut for the deeper TS trusses. Thirty of these were tested and most failed due to tooth withdrawal at joint 1, as in Photo 34.

L - is the single tension strut used with the deeper TS trusses. Fifteen were tested and most failed by tooth withdrawal at joints 1 and/or 3. See Photo 35.

M - is the compression web arrangement for the shallower wood web trusses. Five were tested and all failed by plate buckling at joint 2 as shown in Photo 36.

N - is the same type of specimen as M except that it is used in the deeper WW trusses. Again, five were tested and all failed by plate buckling at joints 1 and/or 2, as shown in Photo 37.

O - is the tension web arrangement used for the shallow WW trusses. Five were tested and all failed by tooth withdrawal at joint 3. See Photo 38.

P - is the tension web shear panel for the deeper WW trusses. As with specimen O, the dominant failure mode was tooth withdrawal at joint 3.

4.5 Effect of Depth

Sixteen different types of shear panels were studied. Half of these were shallow (roughly 300 mm in depth) and the other half were deeper (approximately 400 mm in depth). The experiment was designed in such a way that one could compare behaviours at the two different depths for each type of shear panel specimen. This comparison is shown in Table 21 and the following conclusions can be drawn:

a) the shear stiffness of the deeper specimens is significantly less than that of the shallower specimens (the difference is less marked in the case of the TS specimens), and

b) Specimen A was significantly stronger than specimen E.
Specimen B was significantly stronger than specimen F.
Specimen C was signficantly stronger than specimen G.

In all other cases, there was no significant difference between the strengths of the shallow and deep specimens.

4.6 Superpositions and Equalities

Figures 18 and 19 show the details of the MW type specimens for truss depths 270mm and 397mm, respectively. There are four types shown in each figure. In looking at these four types it's possible to speculate that specimen type C can be thought of as a combination of types A and B. Further, specimen type D might be thought of as a combination of A, B and C. Similarly, when looking at Figures 20 and 21 (which refer to the TS type specimens of the two truss depths), one can think of specimen I as being double specimen J, and specimen K as being double specimen L.

Another speculation of this type relates to Figure 22; if all else is equal, is there an equality between the compression web and the tension web members in WW type trusses? These possible superpositions and equalities are studied in Table 22, with regard to both stiffness and strength. The following observations can be made.

a) Superposition of panel stiffnesses for the MW webs is not appropriate. For both depths, the simple vee is considerably stiffer than the sum of the stiffnesses of the tension strut and the compression strut of that type. On the other hand, for both depths the reinforced vee is much less stiff than the sum of the stiffnesses of the three elements combined into it.

b) Likewise, superposition of strengths does not apply for the MW webs and the discrepancies are in the same directions as those indicated for stiffness.

c) In the case of TS web members superposition is almost achievable, with webs on both sides of the truss being slightly less stiff and strong than double the one-sided web member. It would therefore be conservative and approximately correct to take the properties of a one-sided TS tension strut as being half the value of double-sided tension struts.

d) For WW webs the compression diagonal appears to be much stiffer and much stronger than the tension diagonal with respect to shear properties.

4.7 Comparison of Shear Panels and Trusses

An important point in this project is the extent to which shear panels represent the behaviour of actual trusses. As shown in this project, failure of a truss can take one of three forms:

a) Failure of the splice plate.

b) Breaking of a chord member.

c) Failure of a web or web connection.

The shear panel tests can be used to represent the failure of a web or web connection. The strength of the splice plate can be obtained from the standard tests on truss plates. The strength of the chords can be determined from design values for the strength of wood.

Table 23 examines the failure modes of the trusses and compares them to the failure modes of the appropriate shear panel tests. Generally, where the failure was a result of a web or web connection, the shear panel failure modes were similar to the failure modes of the truss. This was the case in all of the original MW trusses, some of the modified MWD trusses, the modified TSA design, TSB, TSC and TSD modified.

The shear panels cannot be readily used to predict the web and web behaviour in wood web trusses. Since the webs of a wood web truss are continuously joined through the truss plates and the size of the truss plates vary throughout the truss, it is not possible to isolate representative shear panels.

5. MATHEMATICAL MODELS

5.1 General

A major aim of this project was to develop an analytical procedure whereby one can predict both the strength and the stiffness of residential floor trusses, given some information about the properties of the flanges and of the webs. Two approaches are discussed in this chapter. The first one is a pair of simplified methods, based on the shear panel tests, which yielded some success particularly in regard to the metal web trusses. The second technique was to try to adapt a program prepared by Forintek Canada Corp., but this method was not successful.

5.2 Use of the Shear Panel Tests to Predict the Ultimate Load of the Trusses

The shear panel tests described in the previous chapter were designed to simulate a pair of end panels in the floor trusses. Therefore, if one were to take one half of the shear panel test and turn it on its side it could be compared with the results of the truss test.

This calculation is illustrated for the particular case of an MWC truss. Figure 23 shows the test loading of such a truss and a free body diagram of the left end panel of the truss. During the test sixteen concentrated loads were applied to the top chord at a spacing of 290mm, the first one occuring at 146mm from the left reaction of the truss.

The average ultimate load for the five MWC trusses was 3.27kN/m. For a span of 4648mm the average reaction force at failure was 7.60kN and each of the applied forces was 0.95kN.

The two concentrated loads on the top chord at the end panel were distributed to the upper chord panel points by attribution. The first top chord panel point received: $0.95\text{kN} \times \frac{275 + 565}{711} = 1.12\text{ kN}$

The vertical component of the first tension diagonal therefore is:
R - 1.12 = 7.60 - 1.12 = 6.48 kN.

Based on the actual line of action of the web, accompanying this vertical component is a horizontal component of:

$$6.48 \times \frac{260}{359} = 4.69\text{ kN}$$

Looking next at the first bottom chord panel point, there is no external applied vertical force. Therefore the vertical component of the first compression diagonal must be equal and opposite to that in the first tension diagonal, i.e. 6.48kN. Also, the horizontal component of the first compression diagonal is equal to 4.69kN, and is in the same direction as the equal force in the first tension diagonal. Therefore the total horizontal force induced in the bottom chord at that point is 2 x 4.69 = 9.39kN. This is the force V_H shown in Figure 23.

Turning now to the shear panel specimens, type G is the type that corresponds to truss MWC. The twenty original type G specimens had an average failure load per panel of 9.95kN.

The ratio of the shear panel result to the truss result is therefore 9.95/9.39 = 1.06, i.e. the predicted value agreed with the test value within 6%.

These calculations have been carried out for those truss types which were comparable in failure mode to the corresponding shear panels and the results of the comparison are shown in Table 24.

For the MW trusses the predictions are all on the high side, ranging from a low of 2% to a high of 25%. All but one ratio of predicted strength to actual strength is 15% or less; considering the notorious variability of wood this agreement is not that bad.

In contrast, for the TS trusses, all of the predictions are low. The underestimates range from 12% to 33%. By looking at Figures 20 and 21, one can speculate that this underestimate is due to the long length of chord material at the bottom of the specimen which accommodated the displacement transducer. During the tests, however, the tension struts tend to make the two chord members in the centre separate from each other because the bottom horizontal restraints are at a considerable distance below the point where the web members contact the middle chord members. Some bending takes place in the chords and the joint is thereby weakened slightly. It is expected, if the dimension of chord below the metal web pieces was less i.e. the horizontal restraints were closer to the actual joint 3 shown in Figures 20 and 21, that the results for the TS type shear panels would have been higher and therefore the predicted truss ultimate load would have been closer to the actual one.

This observation leads to a recommendation for the development of a standard shear panel test in which the horizontal restraints at the bottom of the specimen are very close to the joint between the web member and the chord member. In the case of the MW specimens these restraints are closer to the joint and the ratios of predicted load to actual load are better than those for the TS type specimens.

5.3 Use of the Shear Panel Tests to Predict the Deflection of the Trusses

In this method, the deflection due to the axial strains in the chords ("bending deflection") and the deflection due to the shear displacements of the shear panels which represent the webs ("shear deflection") are separately calculated and added together to yield the total deflection of the truss. The axial strains and shear displacements are related to midspan deflection by means of a virtual work procedure. Table 25 summarizes the flatwise flexural modulus of elasticity measured for each chord member prior to fabrication. For the virtual work method, it is

assumed that the modulus of elasticity in axial compression or tension is the same as the modulus of elasticity in bending.

This process is illustrated for the particular case of an MWC truss. Figure 24 shows the configuration, joint numbering system, location of the virtual unit load and the horizontal shear forces V_H on a typical web unit.

The average modulus of elasticity values for the chord members were determined as:

top chord: $E = 9650$ MPa

bottom chord: $E = 8270$ MPa

The webs of the MWC trusses corresponded to shear panel type G; the shear stiffness of each vee web unit in a truss is half of the shear panel test value, i.e. 1200 N/mm for the original tests and 1750 N/mm for the modified type G shear panels.

The cross-sectional area A of each chord is $38 \times 89 = 3382$ mm^2.

Table 26 provides the detailed calculations for the determination of the bending and shear deflections. It lists the axial force and the corresponding length of each segment of the chords, and the axial deformation of that chord segment. Placing the virtual unit load on the bottom chord at midspan induces the virtual forces u listed for each chord segment. Multiplying each virtual force by the corresponding axial deformation, and adding their absolute values, the bending deflection of 2.6 mm is obtained.

For each shear panel, the shear force V_H is calculated as the difference in the bottom chord tensions at the base of each vee. The virtual shear force u is calculated in the same way, i.e. it is the difference in the bottom chord virtual forces at the base of the vee. The shear displacement of each panel is calculated as the shear force V_H divided by the shear stiffness of the panel. The values shown in the table were obtained using a shear stiffness of 1750 N/mm. The shear displacements are multiplied by the corresponding virtual forces u, and their sum is the shear deflection of 7.3 mm.

The total deflection is the sum of the bending deflection (2.6 mm) and the shear deflection (7.3 mm) = 9.9 mm which agrees well (particularly if rounded to the nearest mm) with the average test value of 10 mm.

The calculations have been carried out for other trusses and the results are shown in Table 27. The accuracy of the deflection predictions is encouraging in the case of the MW trusses, as it was for strength predictions. However, the stiffness of the TS trusses is considerably underestimated, and this is likely due to the long legs on the shear panel specimens (as discussed previously) which make them more flexible than the webs they are supposed to represent. It is likely that the correspondence between the shear panel behaviour and the performance of TS trusses will improve if the shear panels are modified so that the chords are supported at locations approximating those in the actual truss, i.e. reduce the length of chord below the bottom of the diagonal members.

The above technique was not applicable to the WW trusses, as explained previously. For those trusses, the usual virtual work method was used and the results are listed in Table 28. The underestimate of the prediction is likely caused by not including the contribution of slips in joints other than the bottom chord splice to total deflection.

Returning to a consideration of the TS trusses, another approach could be followed for the determination of web stiffness pending an improvement in the shear panel test specimens. From the behaviour of full-size trusses, one can calculate the stiffness of the webs by:

(a) noting the total deflection of the truss from the short-term test,
(b) calculating the "bending deflection" as described above,
(c) calculating the deflection due to slip in the bottom chord splice, if such a splice is present,
(d) deducting (b) and (c) from (a) to obtain the "shear deflection", and
(e) substituting into the virtual work calculation to obtain the web shear stiffness.

This has been done for four of the truss designs and the values obtained were used to predict the deflections of the other four designs of corresponding type and depth. The results are shown at the bottom of Table 27 for TS trusses, and at the bottom of Table 28 for WW trusses. Agreement between predicted and actual deflections is very good. This technique, therefore, is a satisfactory method of obtaining web stiffnesses but has the disadvantage of having to test entire full-size trusses. Let us think of it as a temporary (although accurate) method until the shear panel specimens are improved.

5.4 The Computer Program SAT by Forintek Canada Corp.

Forintek Canada Corp. was a subcontractor to this project. The terms of reference of their work were as follows:

a) To develop a methodology, supported by computer programs, for the structural analysis of parallel chord trusses. The trusses considered were to have all wood webs, all metal webs, or a combination of the two.

b) To develop full information on the computer programs, including example runs, User's Manual, and discussion of range of applicability.

c) To consider the fundamental property data required, and the manner in which such data could be obtained.

In fulfilling this subcontract Forintek submitted a report entitled "A Methodology for the Analysis of Parallel Chord Trusses". A copy of this report is submitted herewith. Included as a supplement to the Forintek Report was a document entitled "User's Manual for the Program SAT". This document is also being submitted to CHMC.

The input data for the SAT program is cumbersome and time-consuming to generate. In order to facilitate the generation of the data for the SAT program, a second computer program entitled PSAT was developed by Forintek. PSAT is an interactive computer program written in Fortran 77 which will generate the SAT data required, for full size parallel chord trusses, using a simplified truss model.

A detailed account of the input data required for the SAT program is given in the two Forintek reports. Following is a brief summary of the data required for the SAT program and how this information was obtained. The data required for the SAT program can broken down into four general catagories.

a) Data pertaining to the properties of the wooden members.
b) Data pertaining to the metal webs.
c) Information about the connecting plate and teeth.
d) Information about the overall geometry of the assembly being simulated and detailed information about the joint geometries.

Wood properties were taken from the general pool of knowledge available from previous tests on equivalent wood material. In the shear panel tests and the full scale truss tests, the strength of the wood material seldom governed in the strength of the assembly. Therefore, the wood material strength values were not critical to the modelling process. The stiffness of the shear panels and floor trusses were partly dependent on the stiffness of the wood. Wood stiffness values used in the simulation of the full-size trusses were derived from the stiffness value obtained from E-rating the truss chords prior to assembly of the trusses. An average stiffness value was used when modelling the shear specimens.

Data pertaining to the properties of the metal webs were obtained in three manners. The data on the strength and stiffness of the steel material were obtained from published information. The cross-sectional areas, moment of inertia and section modulus were derived by taking detailed measurments of the metal web cross-sections and applying basic engineering formulas. In order to model web buckling, the

out-of-plane eccentricities of the webs has to be determined. The method used to derive this eccentricity was to calibrate the model to test results. Forintek tested the metal webs in compression and used these tests for the calibration. MH used the shear tests for calibration.

One of the unique features of the SAT program is the inclusion of joint slip properties into the overall deformation of the truss. As well, the modelled strength of the assembly can be limited by the strength of the connections. In order to incorporate the joints into the model, information on the load/slip characteristics and strength properties of the plates is required. The properties of the plates were obtained from standard plate tests specified in CSA Standard S347-M1980, "Method of Tests for Evaluation of Truss Plates Used in Lumber Joints". As part of this test series the tooth strength and stiffness values are tested in four directions. These are:

a) Load parallel to wood grain; load parallel to major axis of plate,
b) Load parallel to wood grain; load perpendicular to major axis of plate.
c) Load perpendicular to wood grain; load parallel to major axis of plate.
d) Load perpendicular to grain; load perpendicular to major axis of plate.

From each of these four tests a load vs. slip curve per truss plate tooth can be obtained. These load/slip curves can be described mathematically by the following equation:

$$P = (P_o + P_1 X)\ (1.0 - e^{-(k/Po)})$$

where P is the load/tooth corresponding to the slip X and P_o, P_1 and k are constants derived from the load/slip curve.

The shear specimens were modelled using the geometries shown in Figures 18 to 22. In order to simplify the modelling process, one half of the specimen was used. The loads obtained from the simulation results were then doubled in order to be directly comparable to the test results. The geometries used for the full-scale truss simulations were generated using the PSAT program and the simplified configurations are shown in Figures 2 to 17.

Table 29 gives the pertinent data used in the simulations of the shear specimens along with the results of the simulations. The simulations predicted the strength and stiffness of the shear specimens with variable accuracy. A breakdown of the accuracy of the simulation results in predicting stiffness is given below:

Accuracy of Simulation with respect to Panel Tests	# of Simulations in Category
Within 10%	2
Between 10 and 20%	5
Between 20 and 50%	3
Greater than 50%	6
	16

A breakdown of the accuracy of the simulation results in predicting strength is as follows:

Accuracy of Simulation with respect to Panel Tests	# of Simulations in Category
Within 10%	8
Between 10 and 20%	6
Between 20 and 50%	2
	16

These results indicate that, on the whole, the computer simulation predicted the strength of the shear panels with reasonable accuracy but was a poor predictor of the stiffness of the shear panels.

As indicated previously, the shear panel simulations were used to calibrate the program so that an end eccentricity could be found to predict metal web buckling. These values, along with the other pertinent data given in Table 29 were used in the modelling of the full size trusses. Table 30 shows the results of the SAT simulations on full-size MW trusses. These results indicate that the SAT program, in the form used in this project was not successful in predicting either the strength or stiffness of full size trusses. Further work is being done on improving the results of the SAT simulation. At this time, the outcome from this work looks promising.

5.5 Recommended Analysis Method for MW and TS Trusses

From the preceeding sections, it is concluded that the shear panel method has merit as a method of analysing MW and TS trusses. The following are the steps to be followed in such an analysis for both deflection and strength.

(a) Idealize the truss by a line analogue in which horizontal and vertical members are represented by straight lines along their mid-depths. Diagonal members are represented by straight lines running along their mid-depths in the non-toothed regions (this means that web lines and chord lines may not be concurrent at the joints).

(b) For the given loading, calculate the axial tension or compression forces P in each chord member segment.

(c) Knowing or assuming the modulus of elasticity E of the chords, calculate their axial extension and shortenings Δ_a as:

$$\Delta_a = \frac{PL}{AE}$$

where: L is the length of the chord segment

A is the cross-sectional area of the chord member.

(d) Apply a virtual unit load in the location and the direction of the desired deflection (usually the vertical deflection of the bottom chord at midspan). Calculate the virtual axial forces u in the chords due to the virtual load.

(e) Multiply Δ_a x u for each chord member segment and add them, disregarding sign. The total is the work done due to strains in the chords and is numerically equal to the "bending deflection".

(f) Subdivide the truss into shear panel units. For example, a shear panel unit would be a simple vee or a reinforced vee in the case of an MW truss, and a single or double tension strut plus a wood vertical in the case of a TS truss. A chase in a low-shear area can be ignored (eg. at midspan under symmetrical loading); otherwise, half of the chase is attached to the shear panel unit to its left, and half is attached to the shear panel unit to its right.

(g) Calculate the horizontal shear force V_H acting on the panel.

(h) Determine the shear stiffness k of each panel. If the shear panel test specimens are used to provide this, k is half of the reported stiffness value of the back-to-back test specimen. Alternatively, k can be obtained by calibration from full-size trusses as discussed above. For chases, the appropriate shear panel test specimen consists of a web unit plus half the chord length of the chase, i.e. a long-legged specimen.

(i) For each panel unit, calculate the shear displacement Δ_s as:

$$\Delta_s = \frac{V_H}{k}$$

(j) Calculate the virtual horizontal shear force u on each panel due to the virtual load.

(k) Multiply Δ_s x u for each panel and add them, disregarding sign. The total is the total work done due to shear displacements of the webs and is numerically equal to the "shear deflection".

(l) If a splice exists in the tension chord, determine the force acting on it. From the CSA S347 test results (usually the parallel/parallel test orientation), obtain the slip Δ_{sp} due to the splice tension force. Multiply Δ_{sp} by the virtual tension force u on the splice to obtain the work done due to slip in the splice; this is numerically equal to the "splice displacement".

(m) The total deflection of the truss is the sum of the deflections in (e), (k) and (l). Compare with the permissible deflection.

(n) Check the strength of each shear panel by comparing its horizontal shear force V_H against the ultimate test load obtained in the appropriate shear panel test, taking into account the desired factor of safety.

(o) Check the member stresses in the usual way.

(p) Check the strength of the tension splices in the usual way.

5.6 Comments on the Shear Panels Tests

The usefulness of the shear panel test specimens in predicting both the strength and the deflection of MW trusses provides encouragement to improve the specimens to make it more useful for TS and WW trusses. As mentioned earlier, the future specimens should be proportioned and supported to closely simulate the restraints to the chords which are provided in actual trusses; otherwise, the stiffnesses and strengths will be underestimated.

Consideration might also be given to testing a single web unit, instead of two back-to-back units, provided that an appropriately stiff supporting frame can be fashioned.

Chases can likely be dealt with by testing a specimen which consists of a web unit plus half of the chase. In fact, this specimen will look somewhat like the TS shear panel specimens in Figures 20 and 21, leading to speculation that the low values obtained for these specimens may actually represent the effect of chases adjacent to a web unit in a high-shear area of a truss.

Although the shear panel tests can be used to model moments and shear resulting from joint eccentricity, they do not model the local bending of the chords between panel points. In the trusses used in this project, the distance between panel points was short and therefore, the local bending effects were not considered to be significant. Caution should be used when extrapolating this technique to trusses where there is a longer distance between the panel points.

6. FULL-SIZE LONG-TERM TRUSS TESTS

6.1 Experimental Design

Six of the original twelve truss designs were selected for long-term loading tests. The six that were chosen were the following:

- MW-B
- MW-C
- TS-A
- TS-D
- WW-B
- WW-D

These trusses were chosen according to the following criteria:

a) The spans were all of the same order, i.e. roughly 4900mm.
b) Two different depths of each design type were to be used.
c) A variety of end conditions were to be represented in the tests.

The number of replications was two, i.e. each design was represented by a pair of trusses which were tested together. The total number of trusses tested therefore was twelve.

A major decision which had to be made was the level of load to be applied to the trusses during the long-term tests. On the one hand, a fairly significant level of load was required so that some time-related behaviour would be detected; on the other hand, putting on the full design load for that period of time would be unrealistic. It was therefore decided to use a constant loading level throughout the period of the test equal to the dead load plus ½ of the live load.

The applied loads therefore were the following:

- ½ live load = 0.96 kN/m^2 (20 psf)
- top chord dead load = 0.5 kN/m^2 (10 psf)
- bottom chord dead load = 0.2 kN/m^2 (5 psf)
- total = 1.7 kN/m^2 (35 psf)

For a tributary width of 610mm (2 ft.) the load on a pair of trusses is 2.0 kN/m (140 plf).

The above load level was used on five of the six trusses. The one exception was design WW-B; that was the wooden web truss of the 298 mm depth. For this latter truss, the top chord dead load was taken at 0.2 kN/m^2 (5 psf) leading to a total loading of 1.4 kN/m (30 psf) and to a load for a pair of trusses of 1.7 kN/m (120 plf).

6.2 Materials

The chord and web material used for the long-term tests was the same as that used for the short-term tests on trusses of that design.

6.3 Test Procedure

A special mezzanine floor was built in the MH Research and Development Laboratory in Mississauga. The two trusses of each design type were set up together with plywood sheathing across their top chords, and were supported on a span equal to their maximum design span; these spans were the same as those used in the short-term tests. The two trusses of each pair were braced together at their ends to provide lateral stability. (See Photo 39.)

Double string lines were stretched from one support to the other as a means of measuring deflection of the trusses and paint was sprayed on the joint between the tops of the web members and the chords as an aid in detecting any slip between the two elements.

Masonry blocks were used to apply the load, all of which was applied to the plywood on top of the top chords. This is shown in Photo 40. Care was taken to ensure that there were gaps between adjacent masonry blocks in order to prevent any arching action from taking place between the blocks.

The deflections were read and recorded each day at the same time for the first 30 days under load; thereafter, deflections were measured less often.

6.4 Results

Figure 25 shows the change in deflection of each truss during the first eleven days of loading, and Figure 26 shows the deflections over the first 13 months of load. See also Table 31.

It can be seen from the graph that the trusses experienced a "settling-in" stage during the first month (Dec. 1983) and then remained relatively constant for the next nine months. Two of the trusses actually recovered slightly during the summer. In Oct. 1984 all of the trusses began to move and their deflections have continued to increase through November, December and January of the current winter.

The creep data in this experiment may be compared to those presented by Suddarth, Percival and Comus in their study entitled " Testing and Analysis of 4 x 2 Parallel Chord Metal Plate Connected Trusses". In this study the truss deflections increased quickly each spring and remainded relatively constant over the remainder of the year. A reproduction of their published load-deflection plots appears in Figure 27.

It would be useful to compare the data of the present project with the University of Illinois data but the experiment has not been in progress long enough to do so. The present rapid increase in deflection may correspond with the springtime increases in the Illinois study or, alternatively, it may be an onset of tertiary creep which leads to failure.

A meeting was held of the CMHC Floor Truss Project Technical Advisory Committee on Jan. 22, 1985. The future operation of the long-term truss tests was discussed at this meeting and it was recommended that the trusses be subjected to long-term tests for an additional year. Following that period of recording deformations it is further recommended that the trusses be subjected to short-term loads at that time to determine their residual load-carrying capacity.

7. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

a) Eighty full-size floor trusses were tested under short-term loading. These comprised three different types of trusses:

i) metal vee webs (type MW)
ii) metal "tension strut" diagonals with wood verticals (type TS), and
iii) wood webs (type WW).

For each truss type four different designs were chosen representing two different depths and two different spans. Five replications of each design were tested. The eighty trusses consisted of an original sixty trusses and a later twenty trusses representing five replications each of four modified designs. The information obtained from each truss test consisted of the maximum load, the ratio of the maximum load to the design load, the deflection of the truss at design load and the mode of failure.

b) Two hundred and thirty-three shear panel specimens were tested. These comprised an original sixteen different types of webs used in trusses plus three modified designs. Each test specimen represented two web units from a truss tested back-to-back. Information obtained from each test comprised the ultimate load, the stiffness, i.e. the relationship between applied load and resulting shear deformation, and failure mode. Of the various types of shear panels studied, half were shallow (roughly 300 mm in depth) and the other half were deeper (approximately 400 mm in depth). The experiment was designed in such a way that one could compare behaviours at the two different depths for each type of shear panel specimen.

For the MW type webs superposition does not apply for either stiffness or strength. In the case of TS web members superposition is almost achievable with webs on both sides of the truss being slightly less stiff and strong than double the one-sided web member. It would therefore be conservative and approximately correct to take the properties of a one-sided TS tension strut as being half the value of double-sided tension struts.

c) It was found that the version of the SAT program used in this project did not accurately predict either the strength or stiffness of the full-size trusses. Further work on the program has produced promising results.

d) An analysis method based on the strength of the shear panel test specimens gave reasonably good predictions of the strength of MW trusses, but underestimated the strength of the TS trusses. This was probably due to the legs of the TS shear panel specimen being so long that bending took place and the failure loads in the shear panel specimen were lower than those obtainable in actual trusses.

e) The virtual work approach utilizing the stiffness results from the shear panel test specimens was found to give reasonable predictions of total truss deflection in the case of MW trusses but overestimated deflection in the case of TS trusses. Again this was probably due to the long legs on the TS shear panel test specimens. It is anticipated that better agreement would be obtained if the TS shear panel test specimen were modified to more accurately represent support conditions in a truss.

f) As an alternative, a calibration approach for predicting the deflection of TS and WW trusses was found to work well. However, because it involves the testing of some full size trusses, it is recommended only as a temporary solution until a better TS and WW shear panel specimen can be devised.

g) An analysis method for MW and TS trusses is proposed.

h) Six of the original twelve truss designs were selected for long term loading tests. The number of replications was two, therefore the total number of trusses tested is twelve. They were subjected to a constant loading level throughout the period of the test equal to dead load plus one half of the live load. As of the date of this report the trusses have been under load for over 13 months. The trusses experienced a settling-in stage during the first month and

then their deflections remained relatively constant for the next nine months. In October 1984 all of the trusses began to move and their deflections have continued to increase through November, December and January of the current winter. This recent increase in deflection may be the onset of tertiary creep which leads to failure. It is recommended that the trusses be subjected to long-term tests for an additional year. Following that period of recording information it is further recommended that the trusses be subjected to short-term loads at that time to determine their residual load carrying capacity.

i) Following this project, the following areas were identified as future research needs:

   i) The effect of local bending on the shear panel approach to predict the behaviour of trusses.
   ii) The validity of using a modified shear panel to predict the behaviour of chase openings.
   iii) The use of the shear panel approach on cantilevered trusses.
   iv) The effect of load duration on the analytical approaches presented herein.
   v) The variability in the performance of floor trusses.

MORRISON HERSHFIELD LIMITED

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Table 1 - Summary of Shear Panel Test Results

Truss Type Corresponding To Shear Panel	Specimen Depth (mm)	Specimen Type	Specimen Description	# of Tests	Avg. Initial Stiffness (N/mm)	Ultimate Load (kN)
Metal Web (MW Series)	270	A	Tension Strut	10	2130	10.0
		B	Compression Strut	9	1530	9.6
		C	Simple Vee	20	5010	19.2
		C	Simple Vee - Modified	10	8030	23.5
		D	Reinforced Vee	10	4030	28.4
	397	E	Tension Strut	10	1340	7.3
		F	Compression Strut	10	840	7.0
		G	Simple Vee	20	2400	19.9
		G	Simple Vee-Modified	10	3500	21.4
		H	Reinforced Vee	10	2970	23.0
		H	Reinforced Vee-Mod.	4	3000	28.2
Metal Tension Strut (TS Series)	320	I	Double Tension Strut	30	4410	27.8
		J	Single Tension Strut	15	2320	15.7
	400	K	Double Tension Strut	30	3910	28.1
		L	Single Tension Strut	15	2130	16.3
Wood Web (WW Series)	300	O	Tension Web	5	6390	22.4
		M	Compression Web	5	10000	40.4
	400	P	Tension Web	5	1630	25.4
		N	Compression Web	5	7580	39.5
Total				233		

Table 2 - Shear Panel Specimens Type A

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
A1	2,420	7.48	Wood split under plate 2.
A2	2,100	8.89	Wood split under plate 2.
A3	2,100	>10.60	Wood split under plate 2 & tooth withdrawal at 2.
A4	2,310	8.54	Wood split under plate 2.
A5	1,450	8.10	Wood split under plate 2.
A6	1,890	9.64	Wood split under plate 2.
A7	2,630	14.50	Tooth withdrawal with wood under plate 2.
A8	2,240	11.20	Wood split under plate 2.
A9	1,850	9.90	Tooth withdrawal at 2.
A10	2,300	11.10	Tooth withdrawal with wood under plate 2.
Mean	2,130	10.0	
Std. Dev.	335	2.03	
n	10	10	

Table 3 - Shear Panel Specimens Type B

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
B1	1,730	8.84	Tooth withdrawal at 1 & 2 - 1 web buckled.
B2	1,600	10.4	Tooth withdrawal with wood at 2.
B3	1,751	10.1	Tooth withdrawal at 2.
B4	1,500	10.4	Tooth withdrawal with wood at 1 - 1 web buckled.
B5	1,360	9.90	Tooth withdrawal at 2 - webs buckled.
B6	1,300	9.99	Tooth withdrawal with wood at 2 - 1 web buckled.
B7	1,550	8.62	Wood split under plate at 1.
B8	1,400	8.20	Wood split under plate at 1.
B9	1,620	10.2	Tooth withdrawal with wood at 1.
Mean	1,530	9.63	
Std. Dev.	159	0.84	
n	9	9	

Table 4 - Shear Panel Specimens Type C

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
C1	5,250	18.4	Tooth withdrawal at 3 & web buckling.
C2	3,180	23.3	Tooth withdrawal at 1 & web buckling.
C3	4,200	21.1	Compression tooth withdrawal at 2 & web buckling.
C4	3,500	18.9	Tooth withdrawal at 2 & web buckling.
C5	2,190	14.7	Tooth withdrawal at 3.
C6	5,120	19.4	Web buckling at 2.
C7	4,670	20.2	Web buckling at 2.
C8	5,600	21.6	Tooth withdrawal with wood at 1 & web buckling at 2.
C9	3,720	16.9	Tooth withdrawal at 3 & web buckling.
C10	5,390	19.4	Web buckling at 2.
C11	5,950	19.9	Web buckling at 2.
C12	7,780	21.1	Wood split under plate at 3 and web buckling.
C13	6,230	19.4	Web buckling at 2.
C14	5,950	18.1	Tooth withdrawal at 3 & web buckling.
C15	5,370	20.5	Web buckling at 2.
C16	4,380	16.0	Tooth withdrawal at 3.
C17	6,710	20.9	Wood split under plate at 3 & web buckling.
C18	4,960	19.7	Tooth withdrawal at 3 & web buckling.
C19	5,450	14.4	Tooth withdrawal at 3.
C20	4,580	19.5	Tooth withdrawal at 3 & web buckling.
Mean	5010	19.2	
Std. Dev.	1280	2.26	
n	20	20	

Table 5 - Shear Panel Specimens Type C Modified

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
C21	8,000	24.9	Tooth withdrawal at 3.
C22	8,050	20.5	Tooth withdrawal at 3.
C23	5,600	24.0	Tooth withdrawal at 3.
C24	6,650	26.6	Tooth withdrawal with wood at 3.
C25	6,230	25.2	Wood split under plate at 1.
C26	9,810	26.4	Tooth withdrawal at 3 & web buckling.
C27	9,340	23.6	Tooth withdrawal at 3.
C28	10,070	20.9	Tooth withdrawal at 2 & web buckling.
C29	7,640	19.6	Tooth withdrawal at 3.
C30	8,910	23.0	Tooth withdrawal at 3.
Mean	8,030	23.5	
Std. Dev.	1,530	2.46	
n	10	10	

Table 6 - Shear Panel Specimens Type D

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
D1	4,090	24.4	Wood split under plate and tooth withdrawal at 6 & 7.
D2	4,090	30.1	Wood split under plate at 6 & 7.
D3	3,850	31.9	Wood split under plate at 3.
D4	5,120	27.9	Wood split under plate at 6 & 7.
D5	4,670	27.3	Wood split under plate at 6 & 7.
D6	5,250	32.8	Wood split under plate at 6 & 7.
D7	3,710	31.5	Tooth withdrawal at 7.
D8	3,310	26.4	Wood split under plate at 6 & 7.
D9	3,080	24.9	Wood split under plate at 6 & 7.
D10	3,110	26.7	Tooth withdrawal at 2.
Mean	4,030	28.4	
Std. Dev.	781	2.99	
n	10	10	

Table 7 - Shear Panel Specimens Type E

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
E1	1,400	6.91	Wood split under plate at 2.
E2	1,580	7.79	Tooth withdrawal at 2.
E3	1,350	7.46	Tooth withdrawal with wood at 2.
E4	1,280	6.97	Tooth withdrawal with wood at 2.
E5	1,310	6.07	Tooth withdrawal at 2.
E6	1,460	6.88	Wood split under plate at 2.
E7	1,130	6.63	Tooth withdrawal with wood at 2.
E8	1,400	7.88	Tooth withdrawal at 1.
E9	1,190	8.84	Wood split under plate at 1.
E10	1,250	7.61	Tooth withdrawal at 1.
Mean	1,340	7.30	
Std. Dev.	133	0.78	
n	10	10	

Table 8 - Shear Panel Specimens Type F

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
F1	955	7.13	Tooth withdrawal at 2 & web buckling.
F2	908	7.39	Tooth withdrawal at 2 & web buckling.
F3	1010	7.15	Wood split under plate at 1 & web buckling.
F4	906	7.99	Tooth withdrawal at 2 & web buckling.
F5	742	7.29	Tooth withdrawal at 2 & web buckling.
F6	860	7.44	Wood split under plate at 1 & web buckling.
F7	901	6.27	Tooth withdrawal with wood at 1 & web buckling.
F8	525	5.28	Tooth withdrawal with wood at 2.
F9	805	6.97	Tooth withdrawal at 2 & web buckling.
F10	778	6.62	Web buckling at 1.
Mean	839	6.95	
Std. Dev.	137	0.75	
n	10	10	

Table 9 - Shear Panel Specimens Type G

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
G1	2,770	22.4	Tooth withdrawal at 2 & buckling.
G2	2,100	20.0	Tooth withdrawal with wood at 3 & buckling.
G3	2,480	20.3	Tooth withdrawal with wood at 1 & buckling at 2.
G4	2,330	22.6	Wood failure under plate at 3 & buckling.
G5	2,680	19.4	Wood split under plate at 1 & buckling at 3.
G6	2,680	21.9	Wood split under plate at 3 & buckling.
G7	2,240	18.8	Wood split under plate at 3 & buckling.
G8	2,580	20.6	Tooth withdrawal at 2 & buckling.
G9	2,210	20.7	Tooth withdrawal at 2 & buckling.
G10	2,480	17.7	Tooth withdrawal at 3 & buckling.
G11	2,710	19.4	Tooth withdrawal with wood at 3 & buckling.
G12	2,450	16.6	Tooth withdrawal at 3 & buckling.
G13	2,500	20.9	Tooth withdrawal at 2 & buckling.
G14	2,450	21.6	Wood split under plate at 1 & buckling at 2.
G15	2,330	18.5	Tooth withdrawal at 3 & buckling.
G16	1,850	19.7	Tooth withdrawal with wood at 3 & buckling.
G17	2,380	19.5	Wood split under plate at 3 & buckling.
G18	2,390	17.6	Tension tooth withdrawal at 1.
G19	1,850	19.0	Wood split under plate at 3 & buckling.
G20	2,470	21.0	Web buckling at 2.
Mean	2,400	19.9	
Std. Dev.	254	1.62	
n	20	20	

Table 10 - Shear Panel Specimens Type G Modified

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
G21	3,280	18.4	Tooth withdrawal with wood at 1.
G22	3,290	19.8	Wood split under plate at 3 & buckling.
G23	3,500	20.5	Tooth withdrawal with wood at 1.
G24	3,940	25.2	Tooth withdrawal at 1.
G25	3,790	26.0	Tooth withdrawal at 1.
G26	3,060	19.5	Tooth withdrawal at 3 & buckling.
G27	3,740	20.3	Tooth withdrawal with wood at 3.
G28	3,500	20.4	Tooth withdrawal at 3 & buckling.
G29	3,150	20.0	Tooth withdrawal at 3.
G30	3,740	23.8	Tooth withdrawal at 1.
Mean	3,500	21.4	
Std. Dev.	302	2.61	
n	10	10	

Table 11 - Shear Panel Specimens Type H

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
H1	3,130	21.7	Wood split under plate at 6 & 7.
H2	3,250	18.9	Wood split under plate at 6 & 7.
H3	2,500	21.3	Wood split under plate at 6 & 7.
H4	3,500	31.2	Tooth withdrawal at 4. Buckling at 4, 5, 6 & 7.
H5	3,150	19.4	Wood split under plate at 6 & 7.
H6	2,630	28.7	Wood split under plate at 6 & 7.
H7	2,330	23.6	Wood split under plate at 6 & 7.
H8	2,630	23.5	Wood split under plate at 6 & 7.
H9	3,060	18.0	Wood split under plate at 6 & 7.
H10	3,500	23.8	Wood split under plate at 6 & 7.
Mean	2,970	23.0	
Std. Dev.	417	4.22	
n	10	10	

Table 12 - Shear Panel Specimens Type H Modified

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
H11	----	----	Support problem with specimen - neglected.
H12	3,250	28.6	Tooth withdrawal at 7 & buckling.
H13	3,500	24.2	Tooth withdrawal at 7.
H14	2,130	25.5	Tooth withdrawal at 3, 6 & 7 & buckling at 7.
H15	3,060	34.6	Tooth withdrawal at 7 & buckling.
Mean	2,990	28.2	
Std. Dev.	598	4.63	
n	4	4	

Table 13 - Shear Panel Specimens Type I

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
I1	4,420	26.8	Tooth withdrawal at 1.
I2	3,850	32.1	Tooth withdrawal at 1.
I3	5,840	30.1	Tooth withdrawal at 1.
I4	4,900	30.6	Wood split under plate at 1.
I5	4,240	24.2	Tooth withdrawal at 1.
I6	3,500	26.6	Tooth withdrawal at 1.
I7	3,940	24.8	Wood split under plate at 1.
I8	4,460	29.7	Wood split under plate at 1.
I9	5,120	25.6	Wood split under plate at 1.
I10	4,160	29.7	Wood split under plate at 1.
I11	3,720	26.9	Wood split under plate at 1.
I12	4,310	25.4	Tooth withdrawal at 1.
I13	3,500	26.2	Tooth withdrawal with wood at 1.
I14	3,840	24.2	Tooth withdrawal with wood at 1.
I15	4,670	20.2	Wood split under plate at 1.
I16	3,270	28.2	Wood split under plate at 1.
I17	3,790	30.9	Tooth withdrawal at 1.
I18	5,250	26.4	Tooth withdrawal at 1.
I19	3,500	25.7	Wood split under plate at 1.
I20	5,150	21.0	Tooth withdrawal at 1.

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Table 13 - Shear Panel Specimens Type I (cont'd)

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
I21	3,870	26.8	Tooth withdrawal at 3.
I22	4,200	25.7	Tooth withdrawal at 1.
I23	4,000	26.9	Tooth withdrawal at 1.
I24	4,330	26.8	Tooth withdrawal at 3.
I25	4,520	33.0	Tooth withdrawal at 3.
I26	4,500	28.4	Tooth withdrawal at 1.
I27	5,150	32.0	Tooth withdrawal at 1.
I28	5,600	33.0	Tooth withdrawal at 3.
I29	5,640	33.4	Tooth withdrawal at 1.
I30	5,140	31.8	Tooth withdrawal at 1.
Mean	4,410	27.8	
Std. Dev.	701	3.41	
n	30	30	

Table 14 - Shear Panel Specimens Type J

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
J1	1,950	15.9	Tooth withdrawal at 1.
J2	2,480	17.6	Wood split under plate at 1.
J3	2,420	17.4	Wood split under plate at 1.
J4	2,410	16.4	Tooth withdrawal at 3.
J5	2,330	12.8	Tooth withdrawal at 1.
J6	2,410	16.4	Tooth withdrawal at 1.
J7	2,500	15.2	Tooth withdrawal at 1.
J8	2,400	16.1	Tooth withdrawal at 1.
J9	2,390	15.9	Tooth withdrawal at 1.
J10	1,750	15.0	Tooth withdrawal at 1.
J11	2,630	18.7	Tooth withdrawal at 3.
J12	2,630	16.1	Tooth withdrawal at 3.
J13	2,380	14.2	Tooth withdrawal at 3.
J14	1,910	14.2	Tooth withdrawal at 1.
J15	2,280	13.9	Tooth withdrawal at 3.
Mean	2,320	15.7	
Std. Dev.	257	1.55	
n	15	15	

Table 15 - Shear Panel Specimens Type K

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
K1	4,380	33.2	Tooth withdrawal at 1.
K2	4,040	26.4	Tooth withdrawal at 3.
K3	3,670	25.6	Wood split under plate at 1.
K4	4,200	29.9	Tooth withdrawal with wood at 1.
K5	4,200	35.1	Wood split under plate at 1.
K6	4,380	32.3	Tooth withdrawal with wood at 1.
K7	4,670	30.0	Tooth withdrawal at 1.
K8	3,500	26.6	Tooth withdrawal at 1.
K9	3,890	26.4	Tooth withdrawal at 1.
K10	4,580	28.2	Tooth withdrawal at 1.
K11	3,940	27.4	Tooth withdrawal with wood at 1.
K12	4,440	24.3	Tooth withdrawal at 1.
K13	3,690	29.5	Tooth withdrawal at 1.
K14	3,710	29.2	Wood split under plate at 1.
K15	4,380	26.9	Tooth withdrawal with wood at 1.
K16	3,810	27.0	Tooth withdrawal at 1.
K17	4,160	27.2	Tooth withdrawal at 1.
K18	3,910	28.0	Tooth withdrawal at 1.
K19	4,030	27.2	Tooth withdrawal at 1.
K20	4,380	30.4	Wood split under plate at 1.

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Table 15 - Shear Panel Specimens Type K (cont'd)

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
K21	2,960	22.8	Tooth withdrawal with wood at 1.
K22	3,790	33.6	Tooth withdrawal at 1.
K23	3,230	25.1	Wood split under plate at 1.
K24	3,320	28.4	Wood split under plate at 1.
K25	4,780	23.3	Tooth withdrawal at 1.
K26	3,270	22.2	Tooth withdrawal at 1.
K27	4,470	32.3	Tooth withdrawal at 1.
K28	3,320	30.6	Tooth withdrawal at 1.
K29	3,250	25.7	Tooth withdrawal at 1.
K30	3,090	28.6	Tooth withdrawal at 1.
Mean	3,910	28.1	
Std. Dev.	507	3.20	
n	30	30	

Table 16 - Shear Panel Specimens Type L

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
L1	2,030	14.4	Tooth withdrawal at 1 & 3.
L2	2,170	17.4	Tooth withdrawal with wood at 1 & plate tooth withdrawal at 3.
L3	2,210	15.4	Tooth withdrawal at 3.
L4	2,280	17.4	Tooth withdrawal at 1 & 3.
L5	----	15.0	Tooth withdrawal with wood at 1 & plate tooth withdrawal at 3.
L6	2,390	17.9	Tooth withdrawal at 1.
L7	2,410	17.4	Tooth withdrawal at 1 and plate tooth withdrawal at 3.
L8	2,530	18.4	Tooth withdrawal at 3.
L9	2,280	18.5	Tooth withdrawal at 3.
L10	2,190	18.4	Tooth withdrawal at 3.
L11	1,870	15.6	Tooth withdrawal at 3.
L12	2,040	15.3	Tooth withdrawal at 1 & plate tooth withdrawal at 3.
L13	1,360	15.0	Tooth withdrawal at 3.
L14	2,040	14.3	Tooth withdrawal at 3.
L15	2,060	13.6	Tooth withdrawal at 3.
Mean	2,130	16.3	
Std. Dev.	285	1.70	
n	14	15	

Table 17 - Shear Panel Specimens Type M

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
M1	9,340	41.4	Plate buckling at 2.
M2	9,630	39.1	Plate buckling at 2.
M3	12,100	41.2	Plate buckling at 2.
M4	9,270	41.6	Plate buckling at 2.
M5	9,700	38.6	Plate buckling at 2.
Mean	10,000	40.4	
Std. Dev.	1,180	1.41	
n	5	5	

Table 18 - Shear Panel Specimens Type N

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
N1	6,470	42.0	Plate buckling at 1 and 2.
N2	8,950	37.4	Plate buckling at 1 and 2.
N3	7,240	37.8	Plate buckling at 2.
N4	7,000	40.7	Plate buckling at 2.
N5	8,250	39.8	Plate buckling at 1 and 2.
Mean	7,580	39.5	
Std. Dev.	1,000	1.94	
n	5	5	

Table 19 - Shear Panel Specimens Type O

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
O-1	8,050	21.1	Tooth withdrawal at 3.
O-2	7,960	23.4	Tooth withdrawal at 3.
O-3	4,440	17.6	Tooth withdrawal at 3.
O-4	6,420	26.6	Tooth withdrawal at 3.
O-5	5,060	23.1	Tooth withdrawal at 3.
Mean	6,390	22.4	
Std. Dev.	1,640	3.32	
n	5	5	

Table 20 - Shear Panel Specimens Type P

Specimen #	Initial Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode
P1	1,430	16.9	Tooth withdrawal at 3.
P2	1,680	25.0	Tooth withdrawal at 3.
P3	1,590	24.5	Wood split under plate at 1.
P4	1,460	30.8	Tooth withdrawal at 3.
P5	2,000	29.7	Tooth withdrawal at 3.
Mean	1,630	25.4	
Std. Dev.	229	5.50	
n	5	5	

Table 21

Effect of Depth on Stiffness and Strength of Shear Panel Specimens

Specimen Type	Initial Stiffness (N/mm)		Ultimate Load (kN)	
	Shallow	Deep	Shallow	Deep
MW				
Tension strut	A : 2130	E : 1340	A : 10.0	E : 7.3
Comp. strut	B : 1530	F : 840	B : 9.6	F : 7.0
Simple Vee	C : 5010, 8030	G : 2400, 3500	C : 19.2, 23.5	G : 19.9, 21.4
Reinf. Vee	D : 4030	H : 2970, 3000	D : 28.4	H : 23.0, 28.2
TS				
Double TS	I : 4410	K : 3910	I : 27.8	K : 28.1
Single TS	J : 2320	L : 2130	J : 15.7	L : 16.3
WW				
Tension	O : 6390	P : 1630	O : 22.4	P : 25.4
Comp.	M : 10,000	N : 7580	M : 40.4	N : 39.5

Table 22

Superpositions and Equalities

Comparison	Stiffness (N/mm)	Strength (kN)
MW		
C : A + B	5010, 8030 : 2130 + 1530 = 3660	19.2, 23.5 : 10.0 + 9.6 = 19.6
	5010 = 8670	19.2 = 38.8
D : A + B + C	4030 : 2130 + 1530 + 8030 = 11690	28.4 : 10.0 + 9.6 + 23.5 = 43.1
G : E + F	2400, 3500 : 1340 + 840 = 2180	19.9, 21.4 : 7.3 + 7.0 = 14.3
	2400 = 4580	19.9 = 34.2
H : E + F + G	2970, 3000 : 1340 + 840 + 3500 = 5680	23.0, 28.2 : 7.3 + 7.0 + 21.4 = 35.7
TS		
I : 2J	4410 : 2 x 2320 = 4640	27.8 : 2 x 15.7 = 31.4
K : 2L	3910 : 2 x 2130 = 4260	28.1 : 2 x 16.3 = 32.6
WW		
O_T : M_C	6390 : 10,000	22.4 : 40.4
P_T : N_C	1630 : 7580	25.4 : 39.5

Table 23

Comparison of Failure Modes - Trusses and Shear Panels

Truss		Shear Panel	
Design	Dominant Failure Modes	Shear Panel Type	Dominant Failure Modes
MWA	W	C	W,B
MWB	W, S	D, C	S, W, B
MWC	W, B	G	W, B
MWD	W	H, G	S, W, B
TSA	SP	I, J	S, W
TSB	W	J	W
TSC	W	K, L	W
TSD	-	-	-
WWA	SP	O, M	W, PB
WWB	S, F	O, M	W, PB
WWC	SP	P, N	W, PB
WWD	SP, $T_{\parallel}$	P, N	W, PB
MWA mod.	W, SP	No comparable shear panel	
MWD mod.	W, SP	H, G	S, W, B
TSA mod.	W	I, J	S, W
TSD mod.	W	K, L	W

B : web buckling
F : flexure of chord
PB : plate buckling
S : wood split under the plate
SP : tension splice
T : tension ∥ grain of chord
W : tooth withdrawal

Table 24

Prediction of Ultimate Load of Trusses

Using the Shear Panel Tests

Truss		Shear Panel		
Design	Actual Failure Load (kN/m)	Type	Predicted Truss Failure Load (kN/m)	Predicted / Actual
MW-A	1.72	C	1.75	1.02
		C mod.	2.14	1.25
MW-B	3.60	D	3.81	1.06
MW-C	3.27	G	3.47	1.06
		G mod.	3.73	1.14
MW-D	3.04	H	3.48	1.15
TS-A mod.	4.23	I	3.27	0.77
TS-B	4.55	J	3.06	0.67
TS-C	5.12	K	4.50	0.88
TS-D mod.	3.86	K	3.30	0.85

Table 25

Flatwise Flexural Modulus of Elasticity of the Truss Chords

Truss	Modulus of Elasticity (MPa)			
	Top Chord		Bottom Chord	
	Long Member	Short Member	Long Member	Short Member
MWA - 1	12,300	8,100	8,600	7,800
- 2	9,500	8,300	11,400	12,300
- 3	10,500	8,100	7,900	7,800
- 4	------	-----	-----	-----
- 5	8,600	8,100	5,300	12,300
MWB - 1	6,400	-----	8,600	-----
- 2	6,700	-----	6,600	-----
- 3	9,000	-----	6,900	-----
- 4	7,200	-----	8,700	-----
- 5	11,100	-----	8,600	-----
MWC - 1	9,100	-----	8,800	-----
- 2	9,300	-----	9,400	-----
- 3	9,400	-----	8,800	-----
- 4	12,400	-----	6,300	-----
- 5	7,100	-----	9,500	-----
MWD - 1	7,100	9,800	9,200	10,100
- 2	7,400	6,700	11,200	6,700
- 3	12,100	9,800	8,600	10,100
- 4	7,900	8,300	6,400	6,700
- 5	11,200	9,200	7,600	10,100
TSA - 1	9,000	15,500	8,900	10,500
- 2	10,500	8,800	9,000	10,600
- 3	11,600	12,200	11,100	15,500
- 4	7,800	13,600	10,300	10,500
- 5	10,300	8,800	9,400	10,600

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Table 25 (cont'd)

Flatwise Flexural Modulus of Elasticity of the Truss Chords

Truss	Modulus of Elasticity (MPa)			
	Top Chord		Bottom Chord	
	Long Member	Short Member	Long Member	Short Member
TSB - 1	6,400	-----	9,900	-----
- 2	12,600	-----	11,900	-----
- 3	12,200	-----	12,800	-----
- 4	11,700	-----	11,400	-----
- 5	8,100	-----	12,400	-----
TSC - 1	10,400	-----	8,300	-----
- 2	7,200	-----	10,700	-----
- 3	12,700	-----	10,100	-----
- 4	9,500	-----	11,900	-----
- 5	11,800	-----	13,200	-----
TSD - 1	9,900	11,200	9,900	10,700
- 2	13,200	10,700	8,600	12,200
- 3	10,400	13,600	8,900	11,200
- 4	10,600	13,600	12,400	10,700
- 5	8,200	13,600	9,900	11,200
WWA - 1	6,700	11,600	10,700	9,800
- 2	8,700	9,400	10,400	11,900
- 3	10,700	5,500	9,600	12,300
- 4	10,900	9,400	6,800	12,600
- 5	9,100	10,000	10,900	7,400
WWB - 1	9,600	-----	9,300	-----
- 2	7,500	-----	9,000	-----
- 3	8,800	-----	10,000	-----
- 4	8,500	-----	12,900	-----
- 5	8,200	-----	9,000	-----

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Table 25 (cont'd)

Flatwise Flexural Modulus of Elasticity of the Truss Chords

Truss	Modulus of Elasticity (MPa)			
	Top Chord		Bottom Chord	
	Long Member	Short Member	Long Member	Short Member
WWC - 1	9,300	7,400	9,900	12,000
- 2	12,500	12,300	9,800	10,800
- 3	11,900	9,800	12,300	10,800
- 4	11,100	12,600	9,800	12,000
- 5	11,800	11,900	10,400	10,800
WWD - 6	9,200	11,100	9,400	11,700
- 7	9,700	12,500	9,400	9,100
- 8	12,300	13,600	5,500	10,800
- 9	13,400	11,800	10,000	11,700
- 10	8,800	11,900	11,600	9,100
MWA - 6	9,800	10,000	7,900	13,300
- 7	10,200	11,700	13,100	13,300
- 8	12,000	10,000	12,300	10,300
- 9	13,300	10,800	12,800	18,300
- 10	11,200	10,800	9,200	12,900
MWD - 6	11,400	8,100	14,100	11,000
- 7	9,500	7,700	10,100	10,000
- 8	12,600	11,300	14,000	11,000
- 9	10,700	7,700	10,600	11,100
- 10	11,800	13,200	9,600	9,200
TSA - 6	10,800	9,000	10,200	9,200
- 7	9,300	5,600	8,500	11,000
- 8	9,500	11,400	8,500	7,700
- 9	13,800	11,400	11,400	9,500
- 10	10,500	11,800	13,700	8,800
TSD - 6	11,000	10,000	11,400	14,200
- 7	9,500	12,500	11,800	14,200
- 8	7,700	12,500	8,400	10,000
- 9	5,500	10,000	8,700	14,200
- 10	9,000	14,200	9,200	10,000

Table 26

Calculation of Deflections of MW Truss at Design Load

Member	Axial Force P (kN)	L (mm)	$\frac{PL}{AE}$ (mm)	Virtual Member Force u	$\frac{PL \times u}{AE}$ (mm)
1-3, 12-14	2295	732	0.0515	0.363	0.019 x 2 = 0.038
3-5, 10-12	6054	712	0.1321	1.088	0.144 x 2 = 0.288
5-7, 8-10	8154	711	0.1776	1.814	0.322 x 2 = 0.644
7-8	8790	332	0.0894	2.177	0.195 x 1 = 0.195
2-4, 11-13	4591	712	0.1169	0.726	0.085 x 2 = 0.170
4-6, 9-11	7517	721	0.1938	1.451	0.281 x 2 = 0.562
6-9	8790	1024	0.3218	2.177	0.701 x 1 = 0.701

Bending Deflection = 2.6 mm

Panel	Shear Force V_H (kn)	Δ_s (mm)	Virtual Member Force u	$\Delta_s \times u$ (mm)
1-3-2, 12-14-13	4591	2.623	0.726	1.905 x 2 = 3.809
3-5-4, 10-12-11	2926	1.672	0.726	1.214 x 2 = 2.428
5-7-6, 8-10-9	1273	0.727	0.726	0.528 x 2 = 1.056
7-8	0	0	0	0

Shear Deflection = 7.3 mm
Total Deflection = 9.9 mm

Table 27

Prediction of Deflection of MW and TS Trusses

Using the Shear Panel Tests

Truss Design	Avg. Actual Deflection at Design Load (mm)	Predicted Deflection (mm)	Predicted / Actual
MW-A	21	20	0.95
MW-B	16	16	1.00
MW-C	10	10	1.00
MW-D	18	13	0.72
TS-A	19	27	1.42
TS-B	5	10	2.00
TS-C	7	14	2.00
TS-D mod.	11	20	1.82
Using the calibration technique:			
TS-B (from TS-A)	5	5.5	1.10
TS-D mod. (from TS-C)	11	12.5	1.14

Table 28

Prediction of Deflection of WW Trusses

Using the Virtual Work Method

Truss Design	Avg. Actual Deflection at Design Load (mm)	Predicted Deflection (mm)	Predicted / Actual
WW-A	21	17	0.81
WW-B	12	10	0.83
WW-C	16	12	0.75
WW-D	13	10	0.77
Using the calibration technique:			
WW-B (from WW-A)	12	13	1.08
WW-D (from WW-C)	13	13	1.00

Table 29

Summary of Shear Specimen Simulations

Specimen	Web Properties				Teeth Properties				Simulation Results			Simulation/Test	
	Thickness (mm)	Width (mm)	Buckling Properties EI ($kN.m^2$)	Buckling Properties End Eccentricity (mm)	Direction	P_0 (N)	P_1 (N/mm)	k (N/mm)	Stiffness (N/mm)	Ultimate Load (kN)	Failure Mode *	Stiffness	Ultimate Load
A	2.34	33.8	0.106	1.01	∥/∥	276	117	1190	1700	8.90-9.79	T	0.81	0.89-0.98
					∥/⊥	205	88	630					
					⊥/∥	164	158	1160					
					⊥/⊥	147	154	630					
B	2.34	33.8	0.106	1.01	See Above				1680	8.01-8.90	T	1.12	0.82-0.92
C (original)	2.34	33.8	0.106	1.01	See Above				9210	17.8-19.6	T & B	1.84	0.94-1.03
D	2.34	33.8	0.106	1.01	See Above				7220	33.8-35.6	T & B	1.81	1.17-1.23
E	2.79	39.2	0.326	3.56	See Above				902	7.12-8.01	T	0.69	0.96-1.08
F	2.79	39.2	0.326	3.56	See Above				998	8.90-9.79	T & B	1.19	1.27-1.40
G (original)	2.79	39.2	0.326	3.56	See Above				2850	19.6-2.14	T & B	1.19	0.98-1.07
H (modified)	2.79	39.2	0.326	3.56	See Above				5150	28.5-30.2	T & B	1.72	1.02-1.08
I	2.03	63.5	0.376	2.54	∥/∥	316	134	1400	4780	24.0-26.7	T	1.09	0.86-0.95
					∥/⊥	347	49	1400					
					⊥/∥	298	85	700					
					⊥/⊥	382	70	700					
J	1.02	63.5	0.188	2.54	See Above				3560	12.4-14.2	T	1.55	0.78-0.89
K	2.08	66.0	0.387	2.54	See Above				3900	24.0-26.7	T	1.00	0.86-0.95
L	1.04	66.0	0.194	2.54	See Above				2920	12.4-14.2	T	1.39	0.73-0.84
M	88.9	38.1	3.39	0.00	∥/∥	338	62	1440	7780	32.0-35.6	T & Y	0.78	0.78-0.87
					∥/⊥	327	70	1930					
					⊥/∥	298	60	700					
					⊥/⊥	298	60	700					
N	88.9	38.1	3.39	0.00	See Above				5170	21.4-24.9	Y	3.23	0.54-0.62
O	88.9	38.1	3.39	0.00	See Above				7370	24.9-26.7	T	1.15	1.13-1.21
P	88.9	38.1	3.39	0.00	See Above				4690	28.5-30.2	T & Y	0.62	1.10-1.16

*Failure Modes - T - tooth withdrawal
- B - web buckling
- Y - plate yielding

Table 30

Predictions Using the SAT Program

Deflection at Design Load

Truss Type	Predicted Deflection (mm)	Avg. Test Deflection (mm)	Predicted / Test
MW-A	28	21	1.33
MW-B	24	16	1.50
MW-C	12	10	1.20
MW-D	22	18	1.22
TS-A	--		--
WW-B	21	12	1.75
WW-D	22	12	1.83

Failure Load

Truss Type	Predicted Failure Load (kN/m)	Avg. Test Failure Load (kN/m)	Predicted / Test
MW-A	0.88 - 1.02	1.72	0.51 - 0.59
MW-B	1.75 - 2.04	3.60	0.49 - 0.57
MW-C	2.04 - 2.34	3.27	0.62 - 0.72
MW-D	1.46 - 1.75	3.04	0.48 - 0.58
TS-A	1.31 - 1.53	3.81	0.34 - 0.40
WW-B	1.75 - 2.04	4.74	0.37 - 0.43
WW-D	1.92 - 2.25	3.82	0.50 - 0.59

Table 31

Long Term Test Results: Deflection (mm) due to DL + 1/2 LL

(avg. of 2 tests)

Truss Type	Long Term (13 months)	Immediate	Ratio $\frac{\text{Long Term}}{\text{Immediate}}$
MW-B	24	11.0	2.18
MW-C	14	6.5	2.15
TS-A	25	12.0	2.08
TS-D	12	6.0	2.00
WW-B	16	7.0	2.29
WW-D	16	8.0	2.00

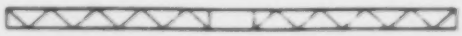
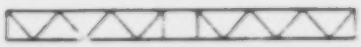
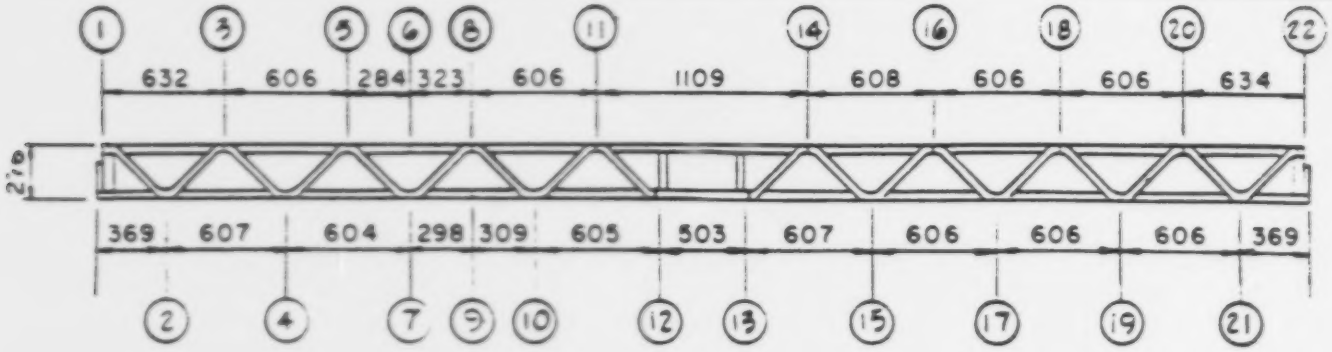
DESIGN	TRUSS DEPTH (mm)	SPAN (mm)	DESIGN LOADING (kN/m)	GENERAL TRUSS CONFIGURATION	DEFLECTION AT DESIGN LOAD	FAILURE LOAD / DESIGN LOAD
MW-A	276	6020	0.80		L/284	2.02
MW-A (MODIFIED)					L/354	3.31
MW-B	276	4521	1.60		L/283	2.25
MW-C	402	4648	1.60		L/465	2.04
MW-D	402	5461	1.60		L/303	1.89
MW-D (MODIFIED)					L/350	1.80
TS-A	320	4932	1.60		L/260	2.37
TS-A (MODIFIED)					L/308	2.63
TS-B	317	3182	1.29		L/636	3.55
TS-C	402	3877	1.60		L/554	3.19
TS-D	402	5002	1.60		L/455	2.99
TS-D (MODIFIED)					L/431	2.41
WW-A	298	5918	1.08		L/282	2.39
WW-B	299	4774	1.46		L/398	3.25
WW-C	397	6268	1.28		L/392	2.45
WW-D	397	5350	1.60		L/412	2.38

FIGURE 1 - AVERAGE RESULTS FOR FULL-SIZE TRUSS TESTS



Truss Span = 6020 mm Design Load = 0.80 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
MWA-1	1.59	19	Web tooth pullout in compression at 3.
MWA-2	1.63	19	Web tooth pullout in tension at 22.
MWA-3	2.03	22	Web tooth/wood pullout in compression at 3.
MWA-4	-	22	Web underpressed at 3.
MWA-5	1.63	24	Web tooth pullout in compression at 3.
Avg.	1.72	21	

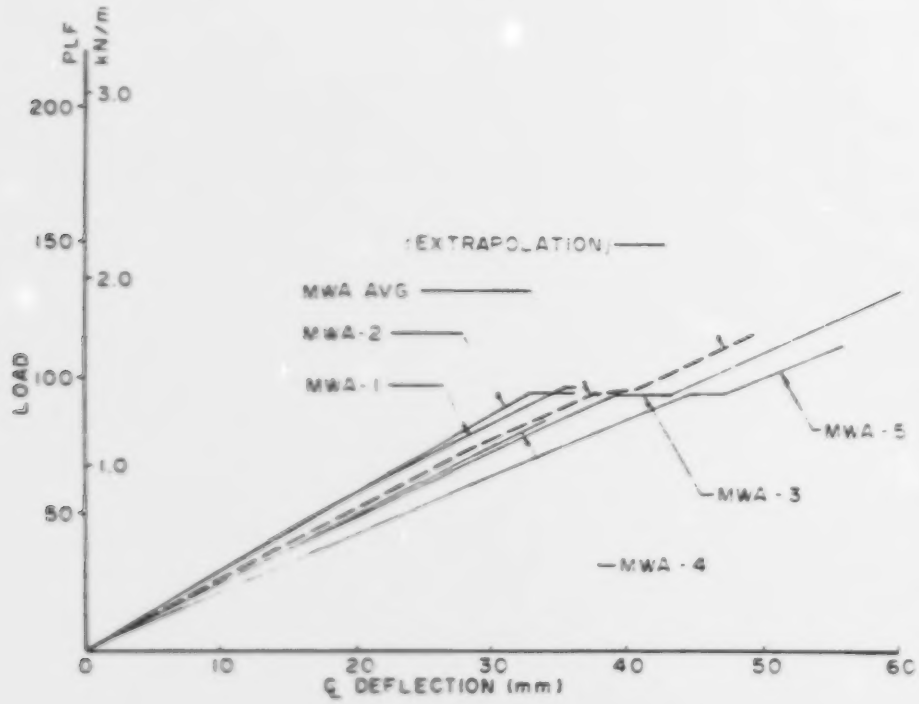
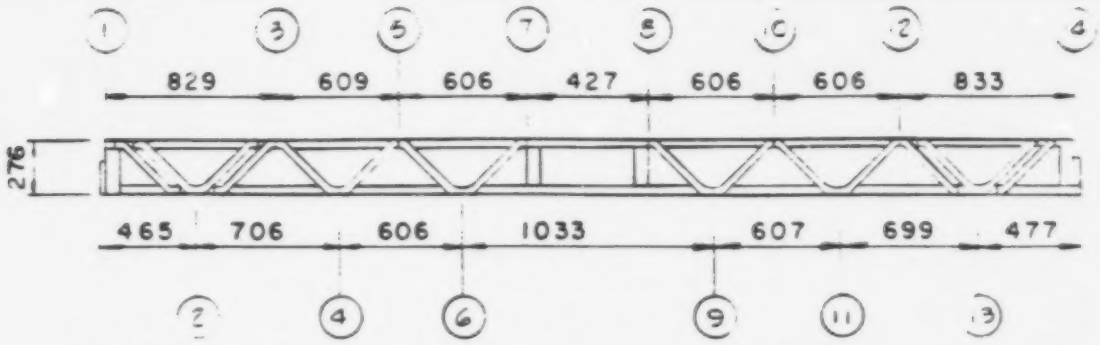


FIGURE 2 - MWA TRUSSES



Truss Span = 4521 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
MWB-1	3.62	17	Outside web tooth/wood pullout in tension at 14.
MWB-2	3.38	14	Wood tension failure at 1.
MWB-3	3.68	17	Wood tension failure at 1.
MWB-4	3.71	16	Outside web tooth pullout in tension at 14.
MWB-5	3.66	15	Outside web tooth/wood pullout in tension at 13.
Avg.	3.60	16	

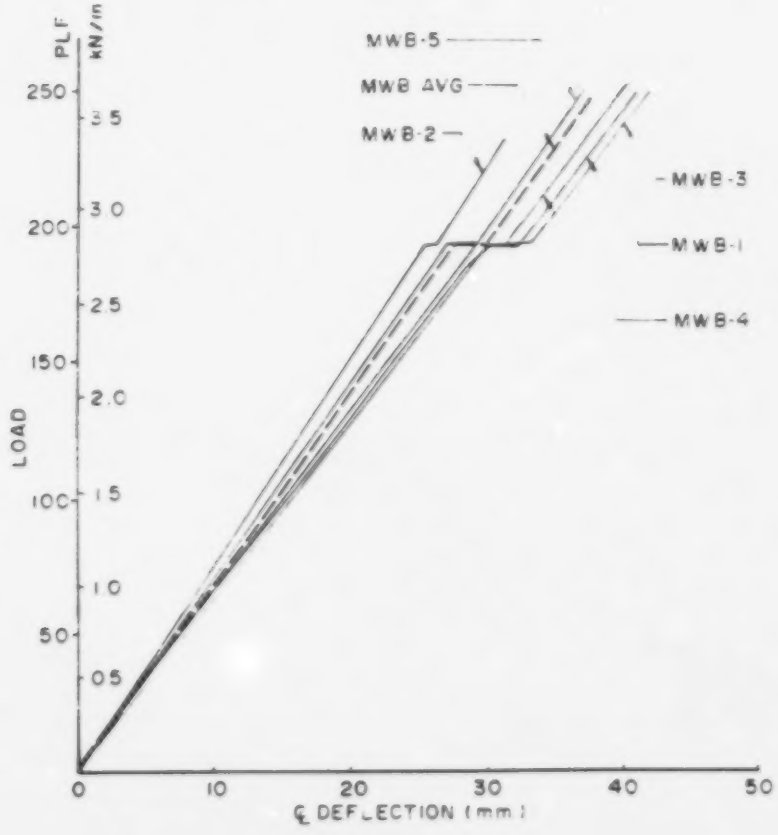
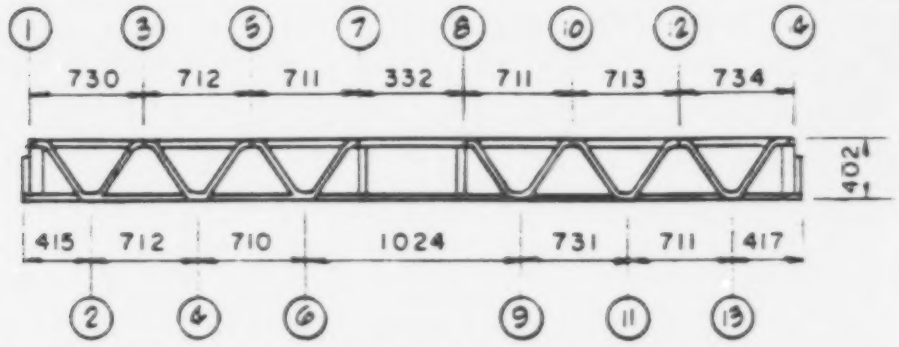


FIGURE 3 - MWB TRUSSES



Truss Span = 4648 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
MWC-1	3.88	11	Buckling of web 2-3
MWC-2	2.77	10	Web tooth/wood pullout in tension at 14.
MWC-3	3.65	10	Compression web buckling and tooth pullout at 12
MWC-4	3.27	8	Compression web buckling and tooth pullout at 12.
MWC-5	2.77	11	Web tooth pullout in tension at 12.
Avg.	3.27	10	

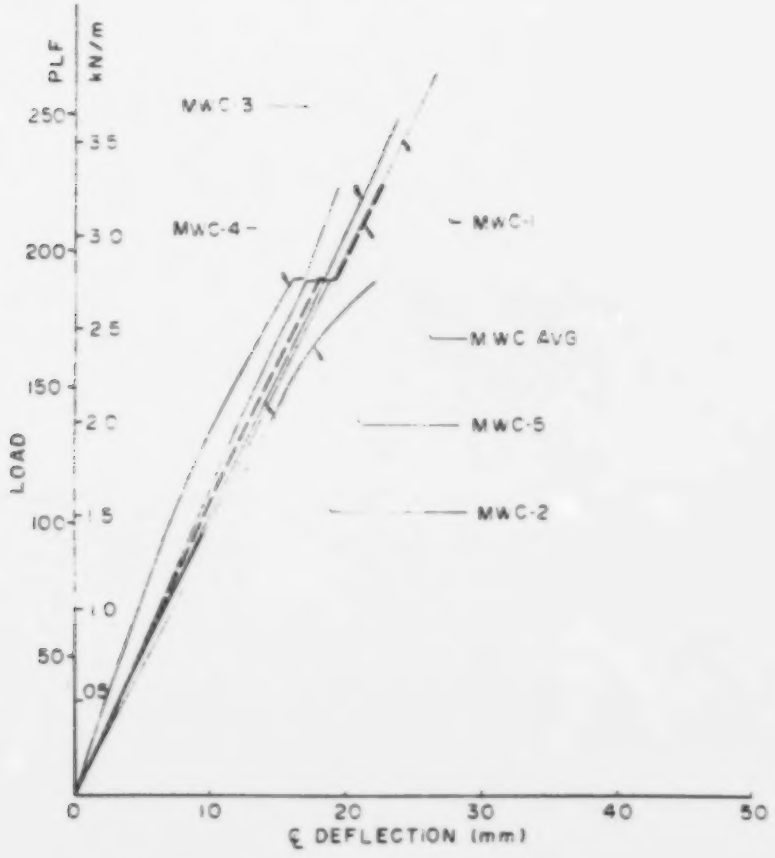
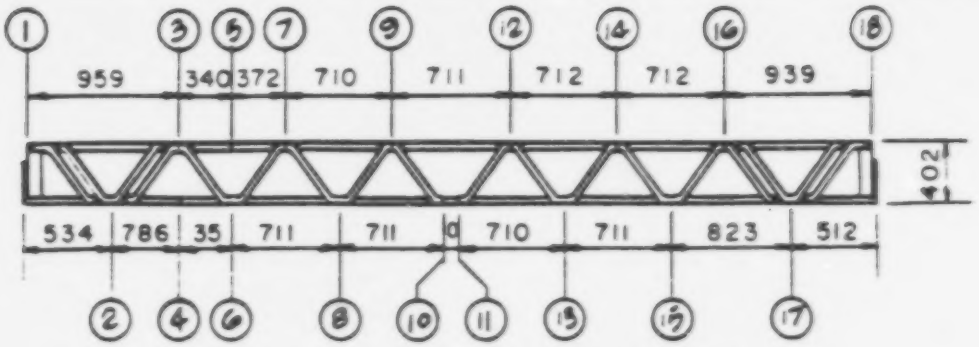


FIGURE 4 - MWC TRUSSES



Truss Span = 5461 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
MWD-1	3.25	17	Web tooth/wood pullout in tension at 3.
MWD-2	2.79	19	Compression web buckling and tooth pullout at 15.
MWD-3	2.77	18	Web tooth/wood pullout in compression at 3.
MWD-4	3.63	18	Compression plate tooth withdrawal at 2.
MWD-5	2.74	18	Web tooth/wood pullout in tension at 3.
Avg.	3.04	18	

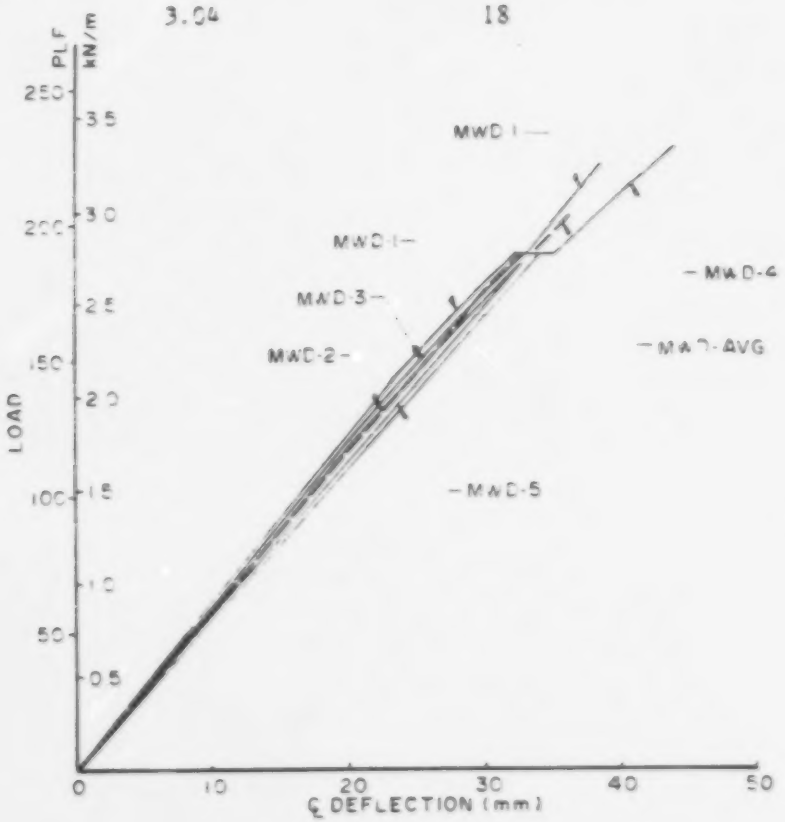
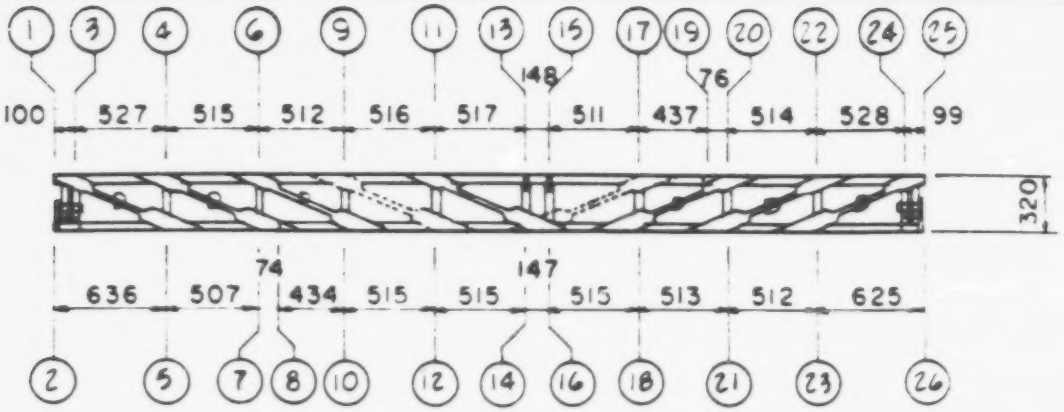


FIGURE 5 - MWD TRUSSES



Truss Span = 4932 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
TSA-1	3.60	19	Bottom chord splice tooth withdrawal.
TSA-2	4.17	19	Bottom chord splice tooth withdrawal.
TSA-3	4.17	18	Bottom chord splice tooth withdrawal.
TSA-4	3.31	20	Bottom chord splice tooth withdrawal.
TSA-5	3.79	19	Bottom chord splice tooth withdrawal.
Avg.	3.81	19	

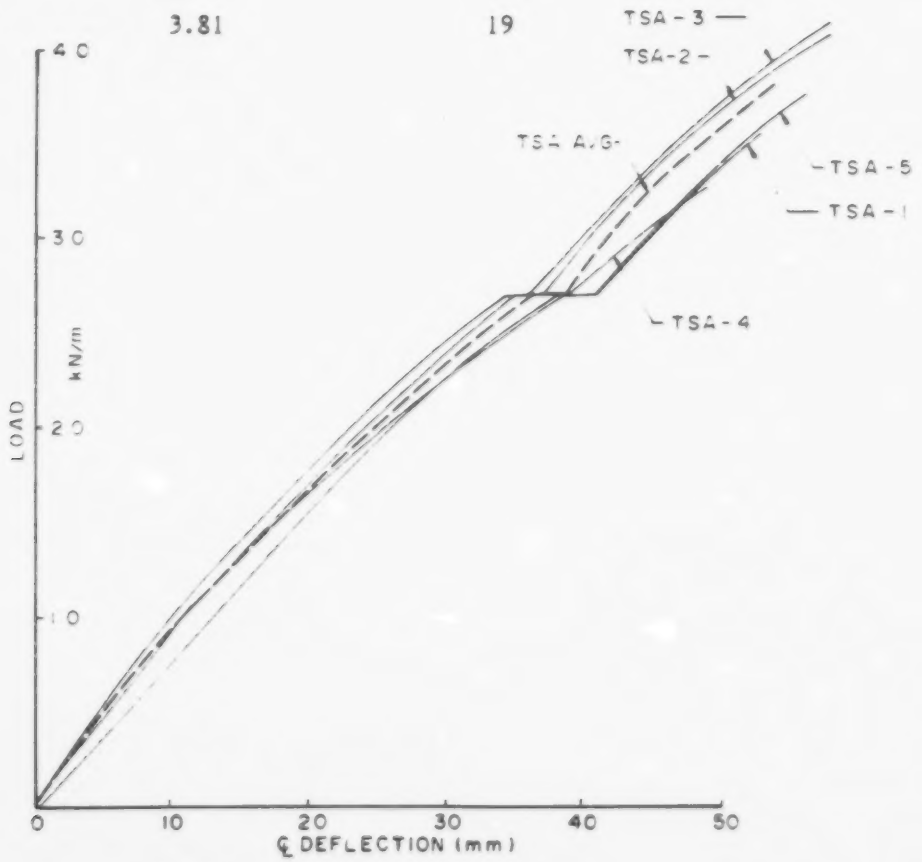
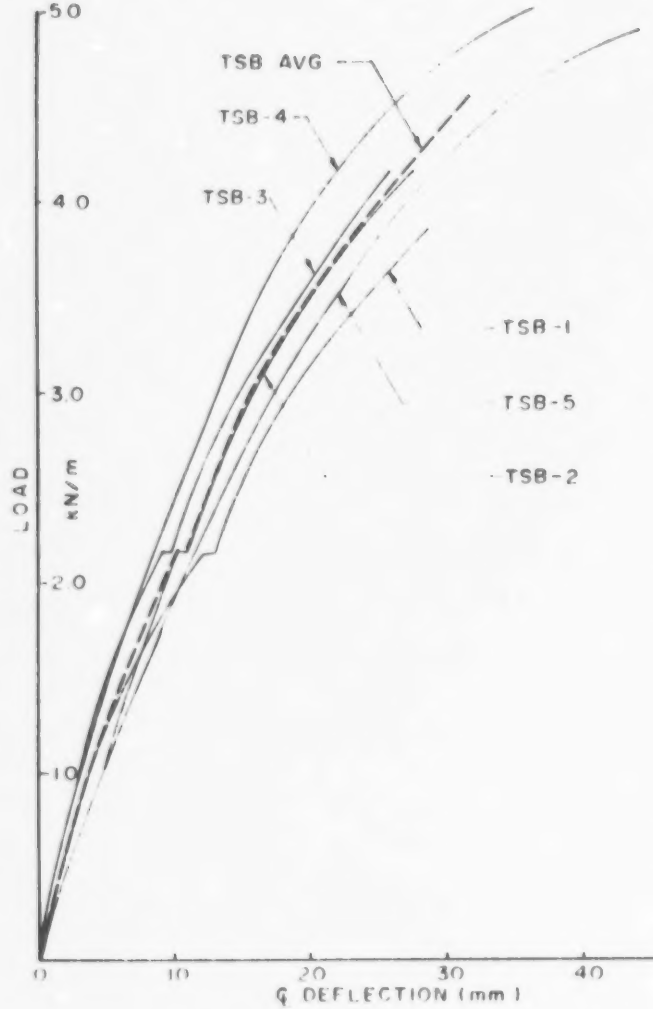
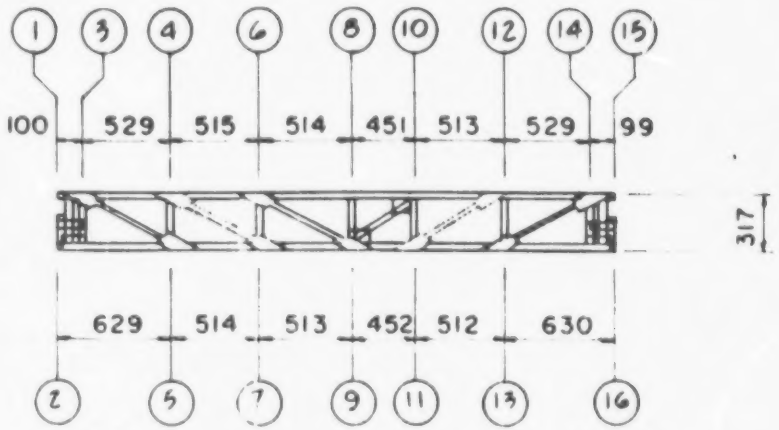


FIGURE 6 - TSA TRUSSES

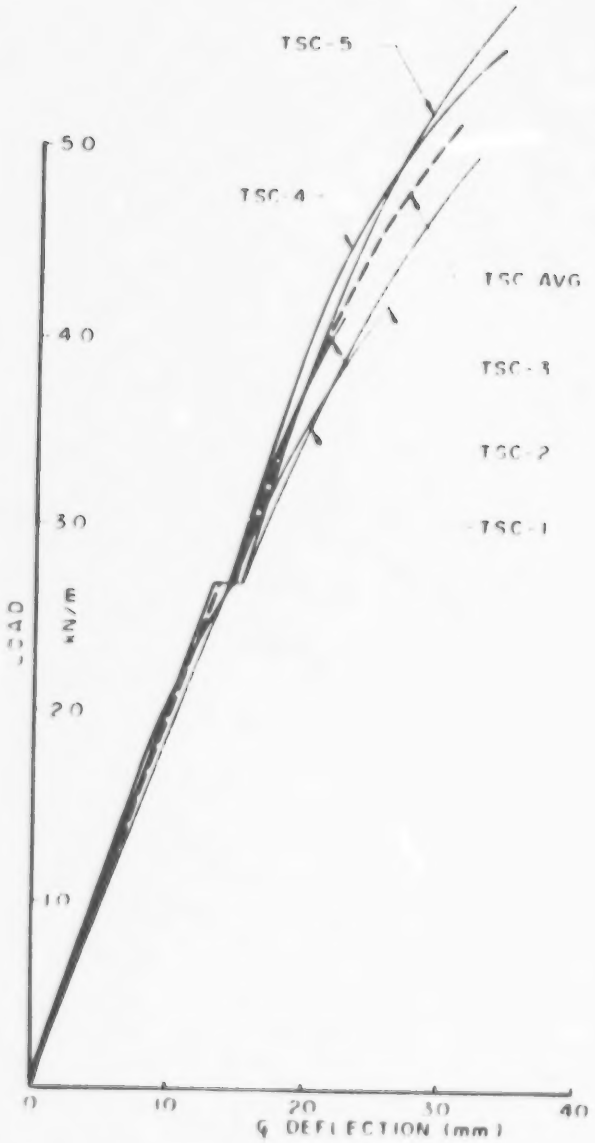


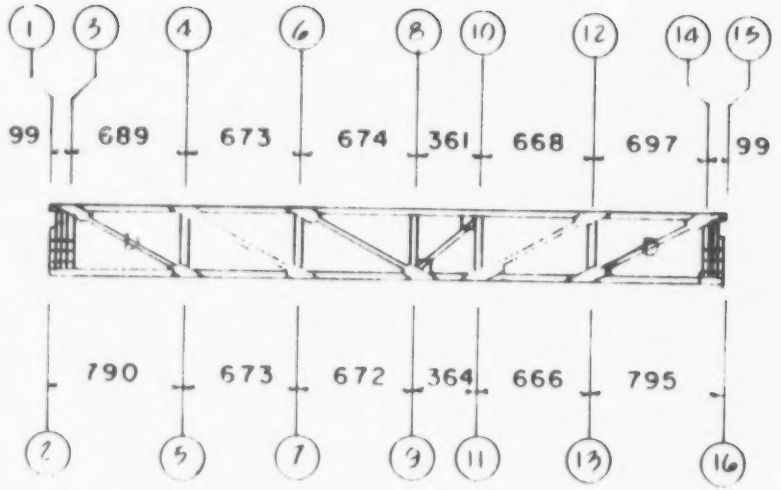


Truss Span = 3182 mm Design Load = 1.28 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
TSB-1	3.88	5	Tension Web Tooth Pullout at 14.
TSB-2	4.46	5	Tension web tooth pullout at 5.
TSB-3	4.46	5	Tension web tooth pullout at 13.
TSB-4	5.03	4	Tension web tooth pullout at 13.
TSB-5	4.93	6	Tension web plate failure at 3.
Avg.	4.55	5	

FIGURE 7 - TSB TRUSSES

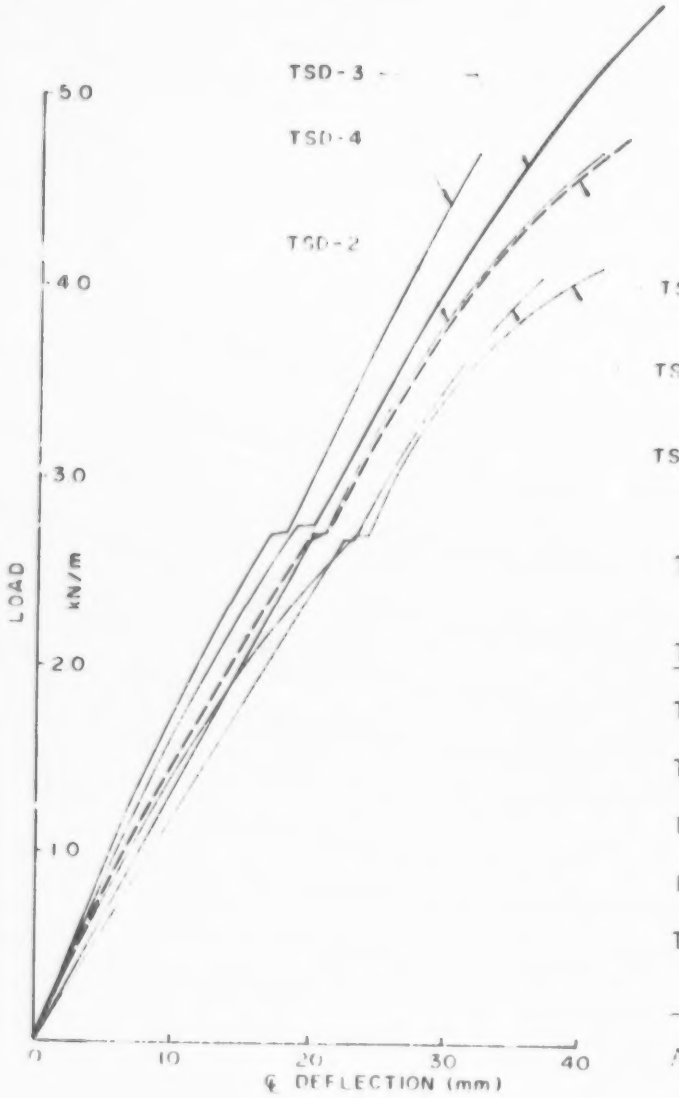


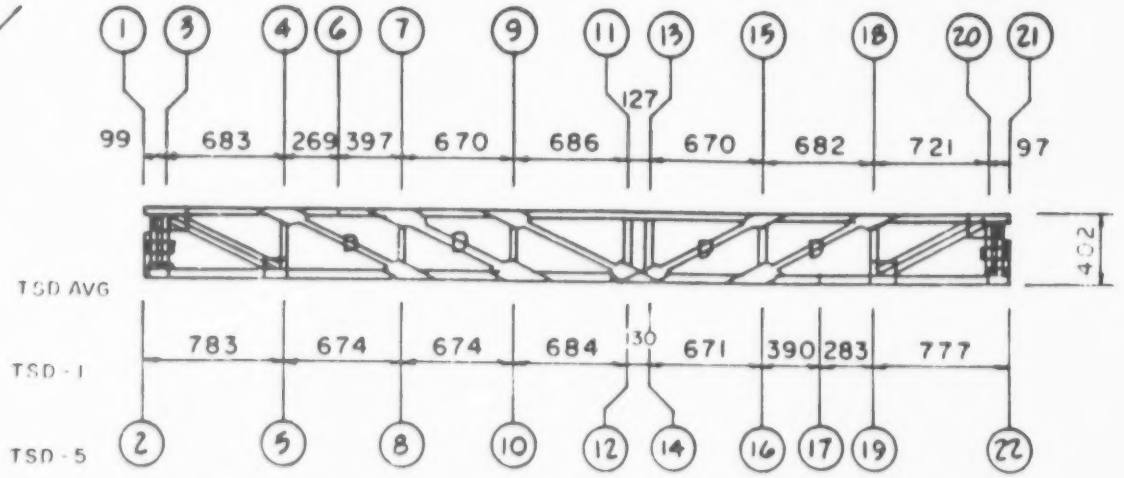


Truss Span - 3877 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
TSC-1	5.00	8	Tension web tooth pullout at 14.
TSC-2	4.14	7	Tension web tooth pullout at 14.
TSC-3	4.44	7	Tension web tooth/wood pullout at 14.
TSC-4	5.57	8	Tension web tooth pullout at 3.
TSC-5	6.44	7	Tension web tooth pullout at 14.
Avg.	5.12	7	

FIGURE 8 - TSC TRUSSES

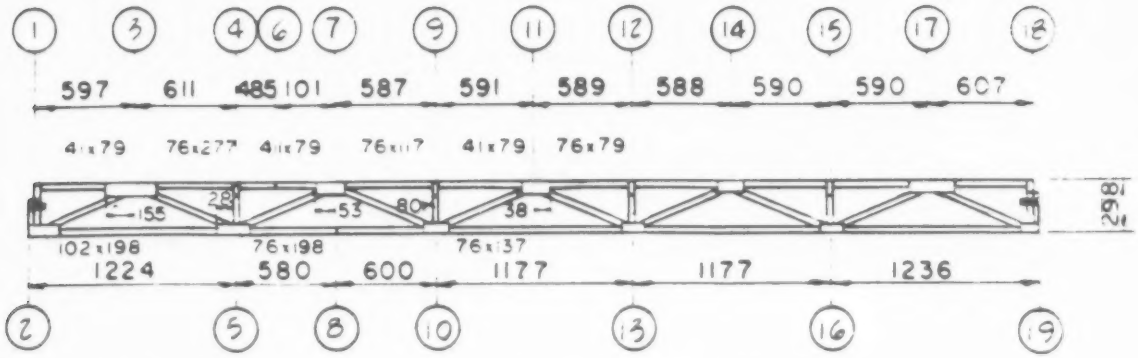




Truss Span = 5002 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
TSD-1	4.17	12	Plate tooth withdrawal from bottom chord at 19.
TSD-2	4.73	12	Bottom chord splice tooth withdrawal.
TSD-3	5.98	16	Plate tooth withdrawal from bottom chord at 5.
TSD-4	5.03	9	Plate tooth withdrawal from bottom chord at 19.
TSD-5	4.09	12	Plate tooth withdrawal from bottom chord at 5.
Avg.	4.80	11	

FIGURE 9 - TSD TRUSSES



Truss Span = 5918 mm Design Load = 1.08 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
WWA-1	2.48	23	Bottom chord tension between 10 & 13.
WWA-2	2.68	19	Bottom chord splice in tension.
WWA-3	2.48	24	Bottom chord splice in tension.
WWA-4	2.64	19	Bottom chord splice in tension.
WWA-5	2.64	19	Bottom chord splice in tension.
Avg.	2.58	21	

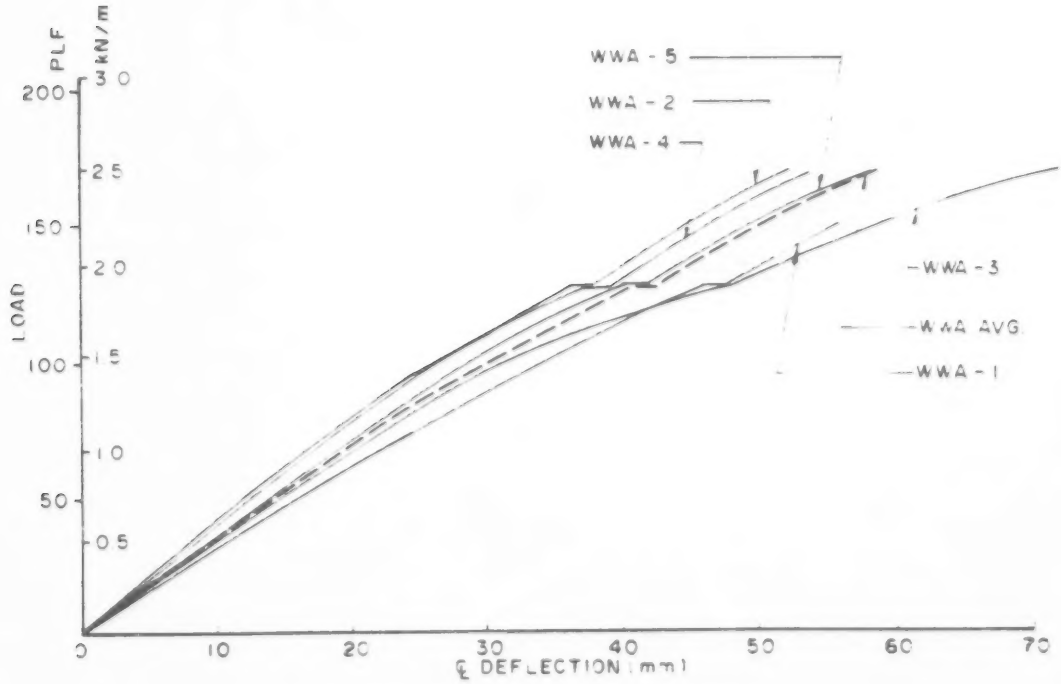
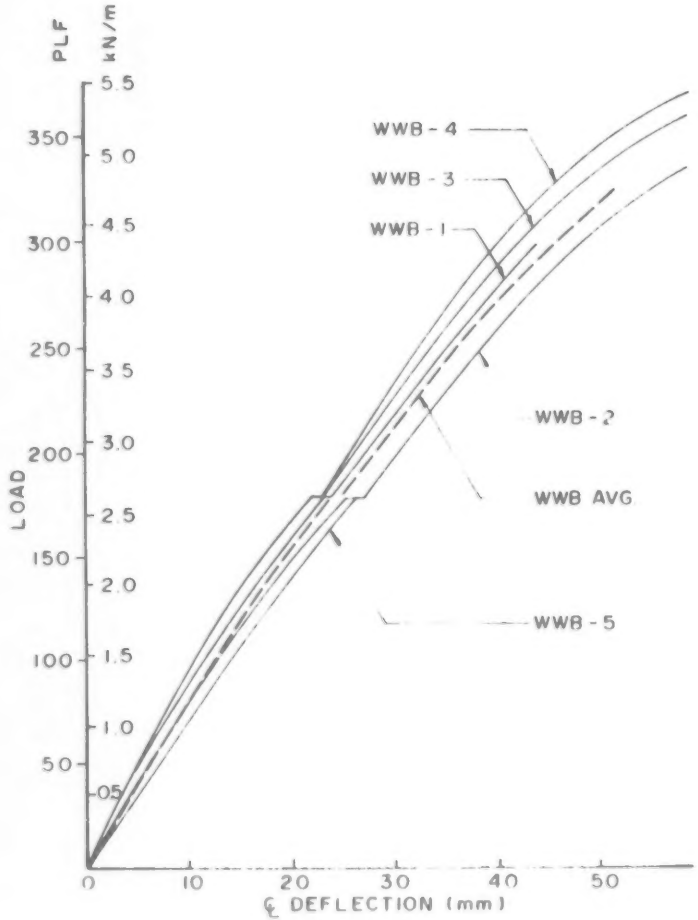
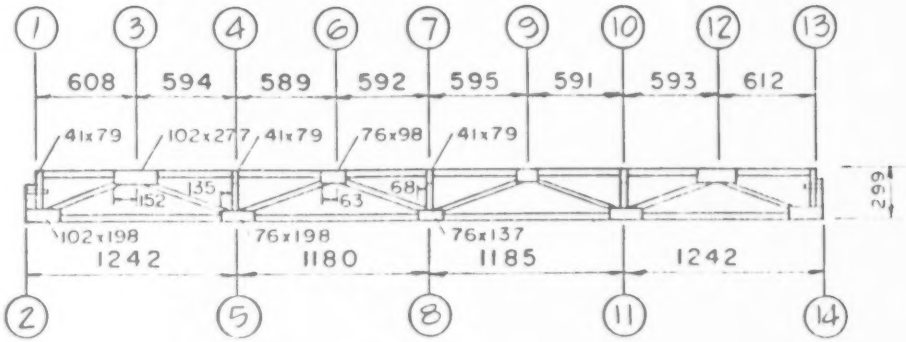


FIGURE 10 - WWA TRUSSES

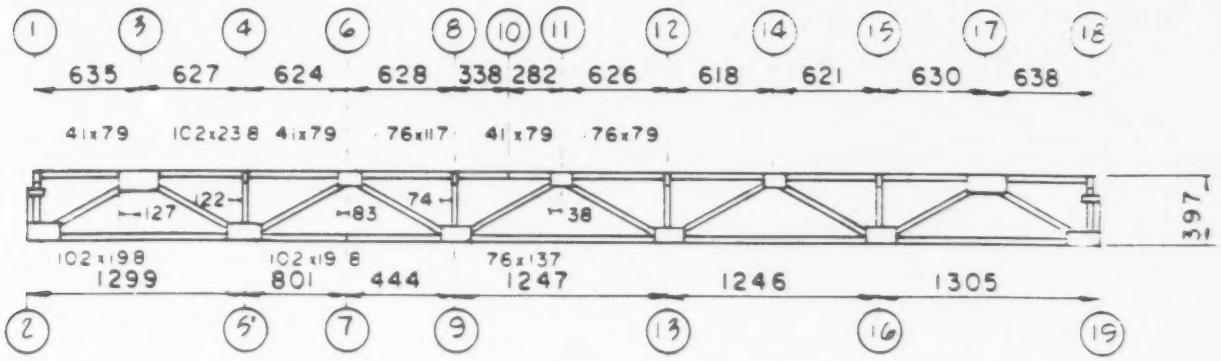




Truss Span = 4774 mm Design Load = 1.46 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
WWB-1	4.67	11	Bottom chord wood/ plate failure at 5.
WWB-2	5.25	13	Bottom chord wood/ plate failure at 11.
WWB-3	5.46	11	Bottom chord wood/ plate failure at 11.
WWB-4	5.69	12	Top chord bending between 9 & 10.
WWB-5	2.63	14	Bottom chord bending between 8 & 11.
Avg.	4.74	12	

FIGURE 11 - WWB TRUSSES



Truss Span = 6268 mm Design Load = 1.28 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
WWC-1	2.51	17	Bottom chord tension between 9 & 13.
WWC-2	3.28	17	Bottom chord splice in tension.
WWC-3	3.38	15	Bottom chord splice in tension.
WWC-4	3.38	14	Bottom chord splice in tension.
WWC-5	3.22	15	Bottom chord splice in tension.
Avg.	3.15	16	

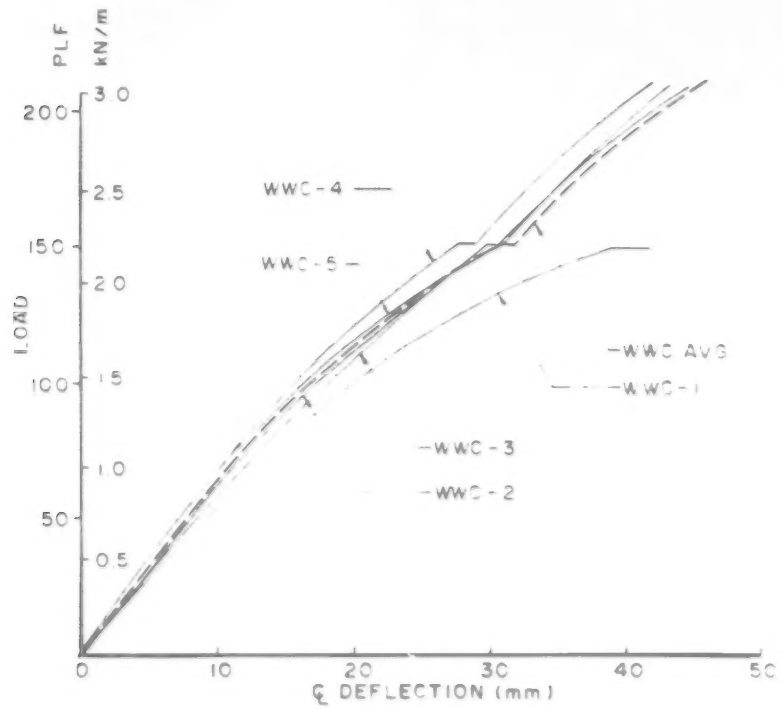
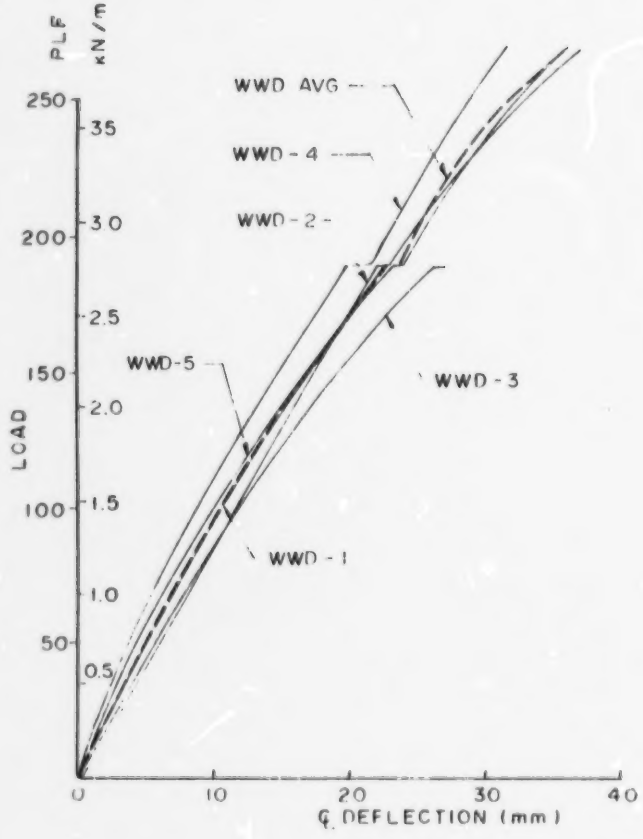
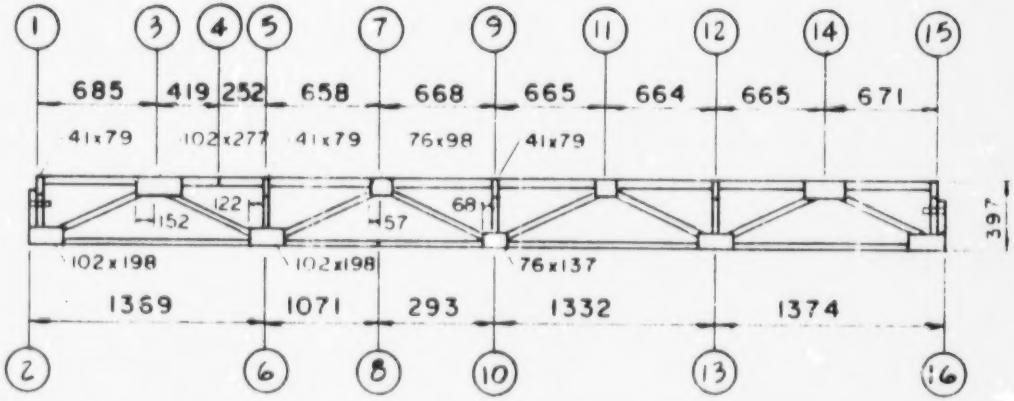


FIGURE 12- WWC TRUSSES

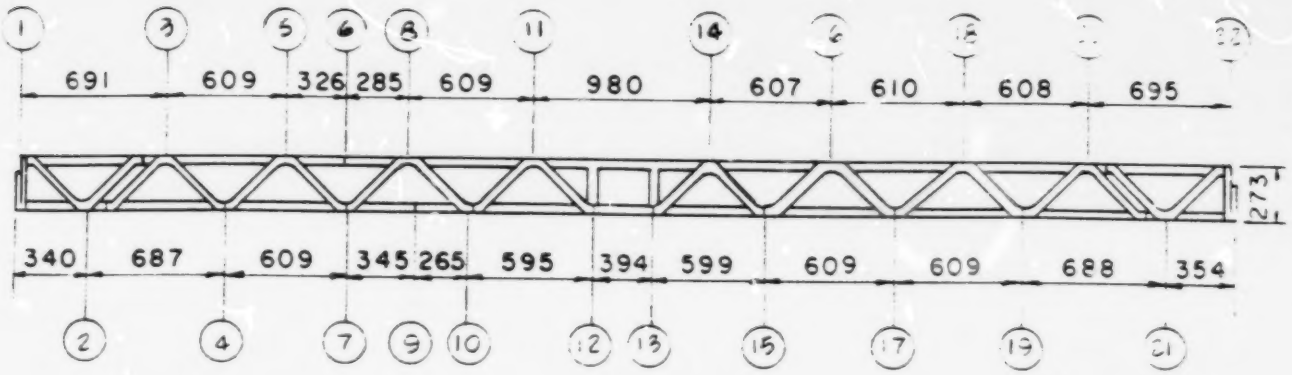




Truss Span = 5350 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
WWD-1	3.65	13	Bottom chord tension between 10 & 13.
WWD-2	4.10	13	Bottom chord splice in tension.
WWD-3	3.06	14	Bottom chord tension between 10 & 13
WWD-4	4.19	11	Bottom chord splice in tension.
WWD-5	4.09	12	Bottom chord splice in tension.
———	———	—	
Avg.	3.82	13	

FIGURE 13 - WWD TRUSSES



Truss Span = 6020 mm Design Load = 0.80 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
MWA-6	2.52	18	Bottom chord tension ⊥ to grain at 4.
MWA-7	2.84	16	Bottom chord splice in tension.
MWA-8	2.52	20	Web tooth pullout in tension at 22.
MWA-9	2.55	13	Web wood/tooth pullout in tension at 22.
MWA-10	2.84	18	Web wood/tooth pullout in tension at 22.
———	———	—	
Avg.	2.65	17	

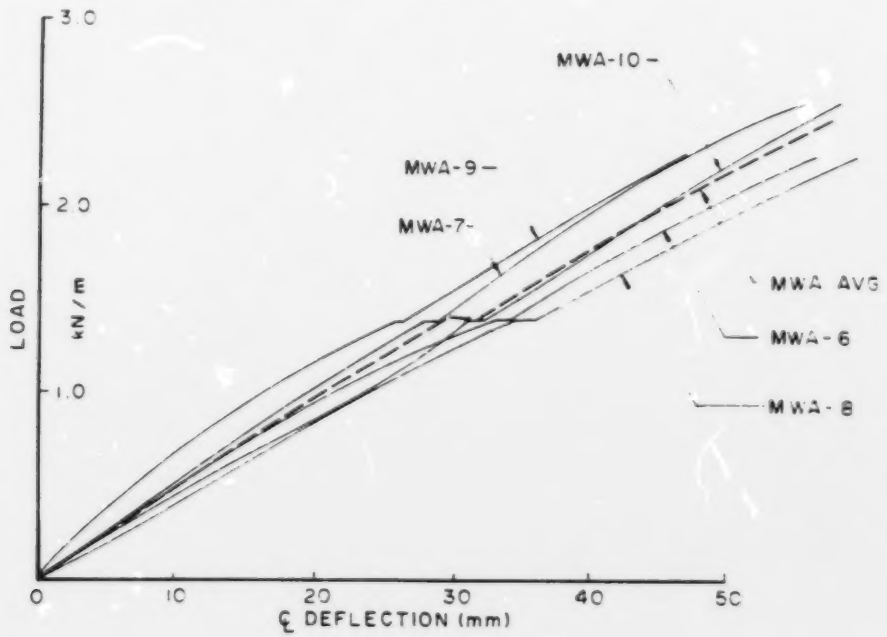
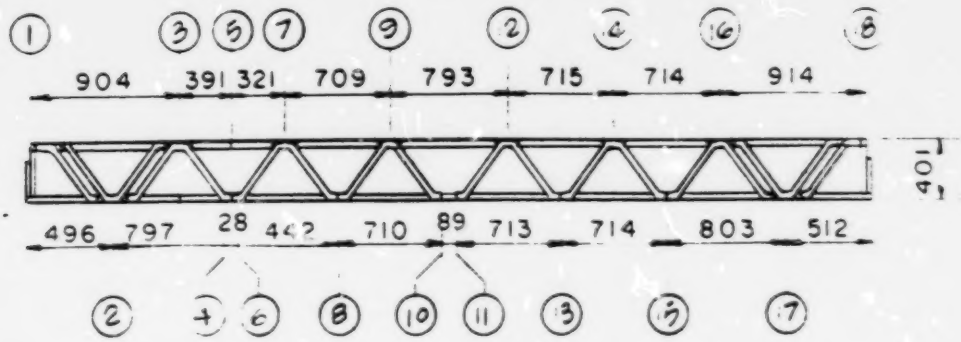


FIGURE 14 - MWA MODIFIED TRUSSES



Truss Span = 5461 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
MWD-6	2.99	15	Web tooth/wood pull[illegible] tension at 18.
MWD-7	3.06	16	Web tooth/wood pullout in tension at 18.
MWD-8	2.77	15	Bottom chord splice tooth withdrawal.
MWD-9	(2.54)	17	Compression web at 16 underpressed leading to plate pullout.
MWD-10	2.77	17	Compression web plate pullout at 6.
———	———	—	
Avg.	2.90	16	

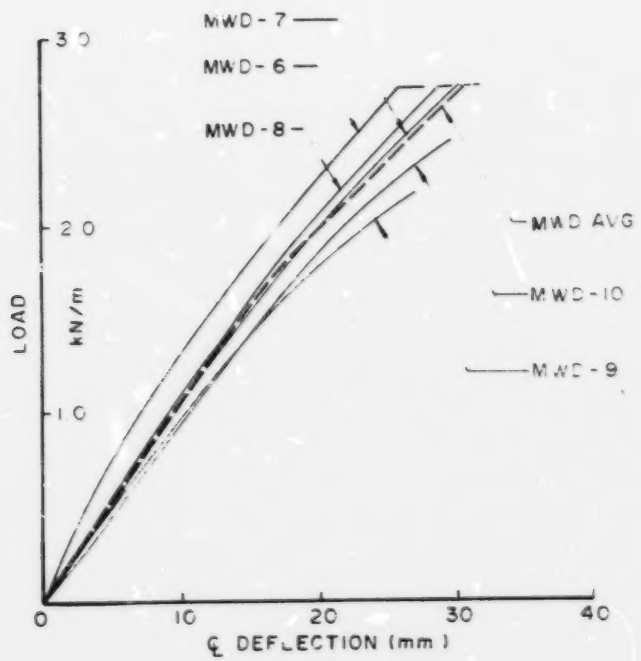
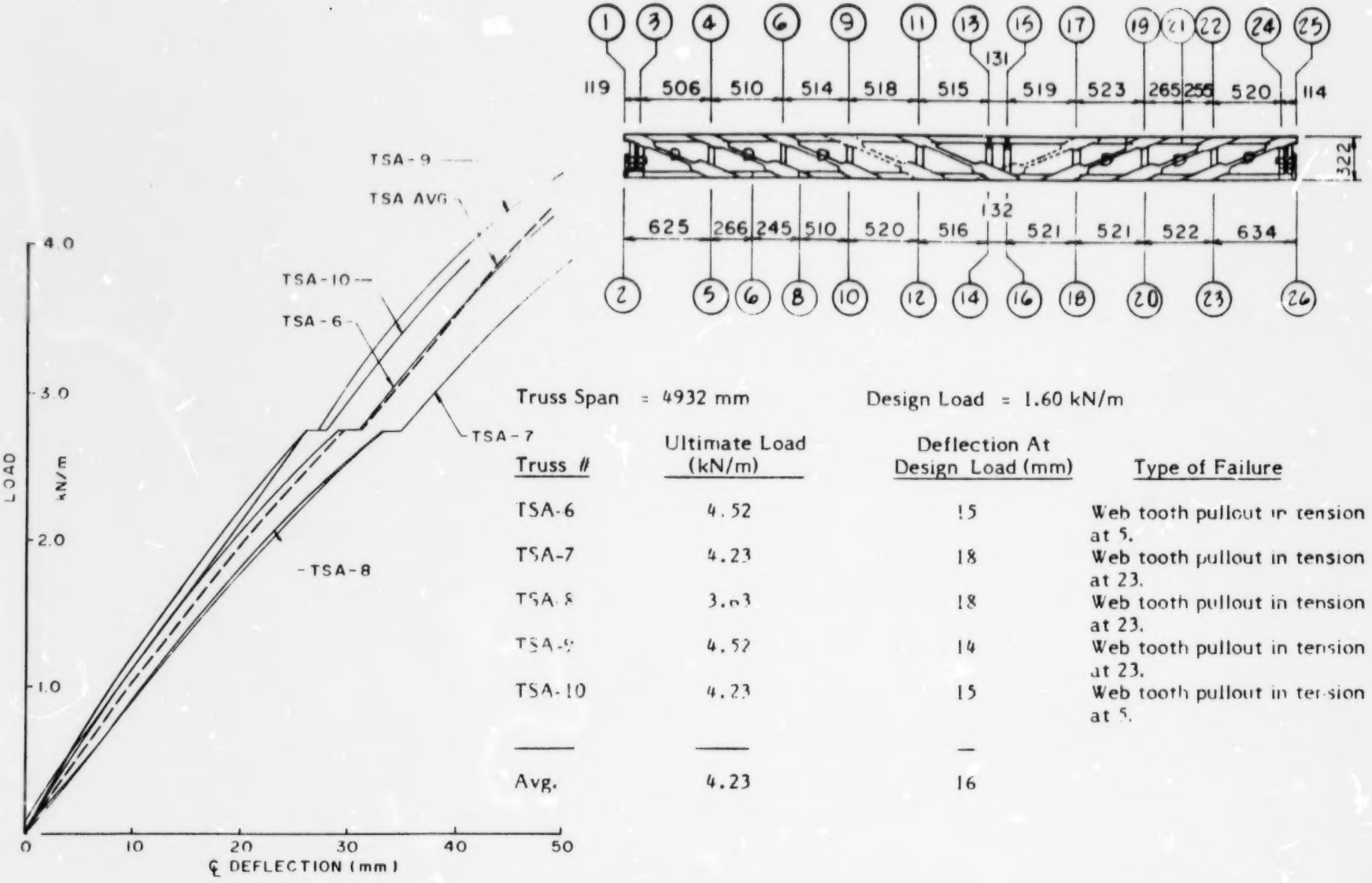


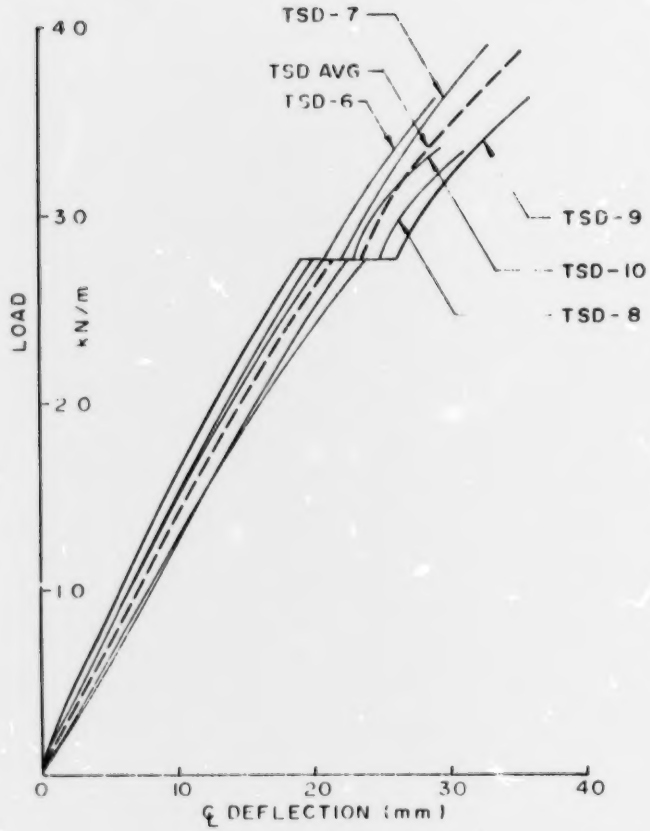
FIGURE 15 - MWD MODIFIED TRUSSES

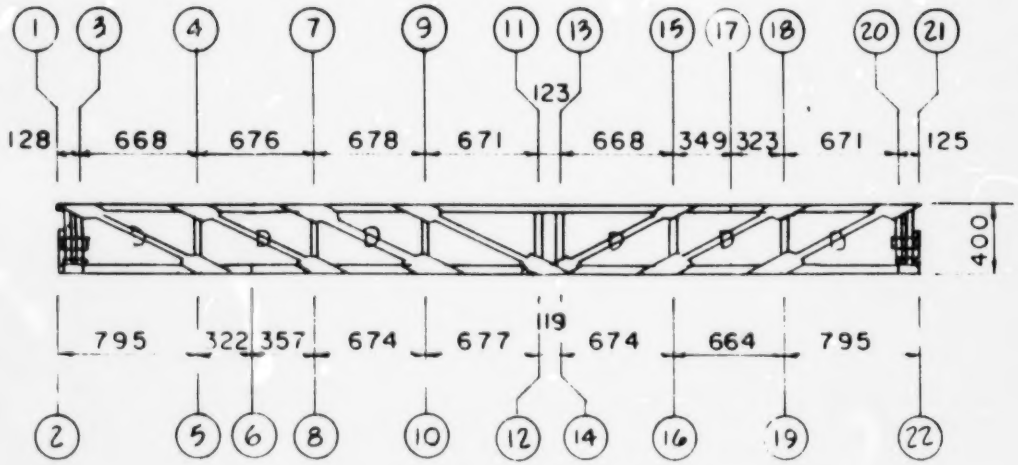


Truss Span = 4932 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
TSA-6	4.52	15	Web tooth pullout in tension at 5.
TSA-7	4.23	18	Web tooth pullout in tension at 23.
TSA-8	3.83	18	Web tooth pullout in tension at 23.
TSA-9	4.52	14	Web tooth pullout in tension at 23.
TSA-10	4.23	15	Web tooth pullout in tension at 5.
Avg.	4.23	16	

FIGURE 16 - TSA MODIFIED TRUSSES

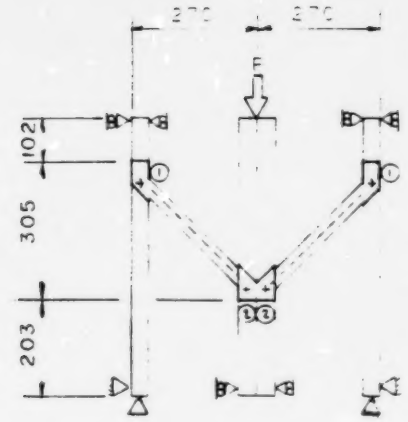




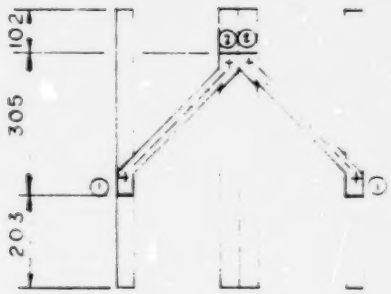
Truss Span = 5002 mm Design Load = 1.60 kN/m

Truss #	Ultimate Load (kN/m)	Deflection At Design Load (mm)	Type of Failure
TSD-6	3.94	10	Web tooth/wood pullout in tension at 18.
TSD-7	4.23	11	Web tooth/wood pullout in tension at 18.
TSD-8	3.65	13	Web tooth/wood pullout in tension at 18.
TSD-9	3.94	13	Web tooth/wood pullout in tension at 18.
TSD-10	3.56	11	Web tooth/wood pullout in tension at 18.
———	———	—	
Avg.	3.86	12	

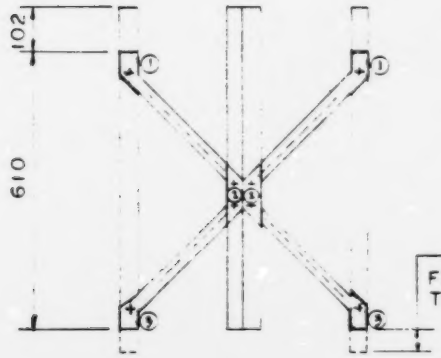
FIGURE 17- TSD MODIFIED TRUSSES



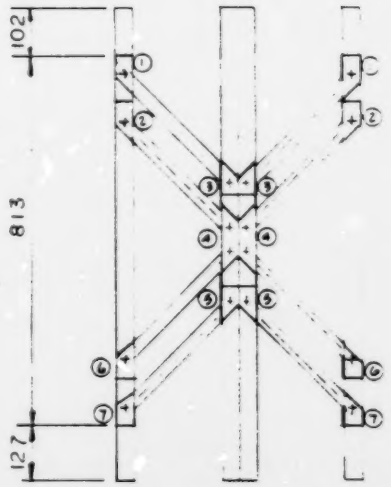
SPECIMEN TYPE A TENSION STRUT



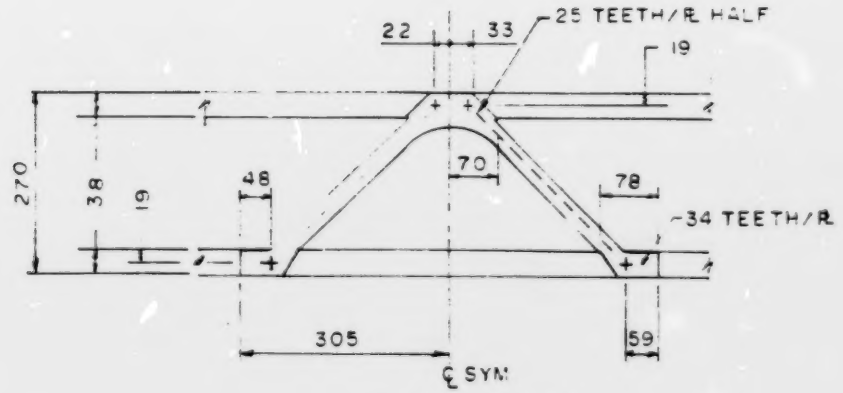
SPECIMEN TYPE B COMPRESSION STRUT



SPECIMEN TYPE C SIMPLE VEE

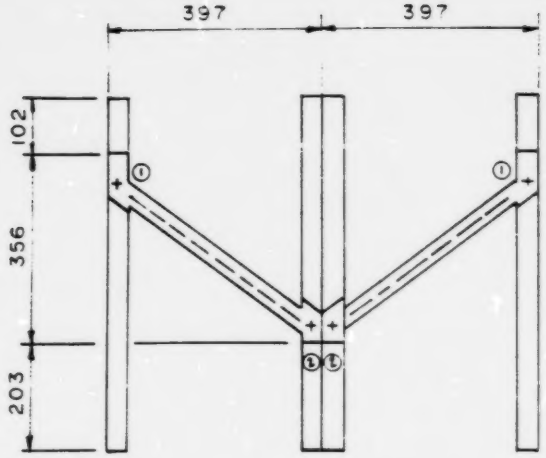


SPECIMEN TYPE D REINFORCED VEE

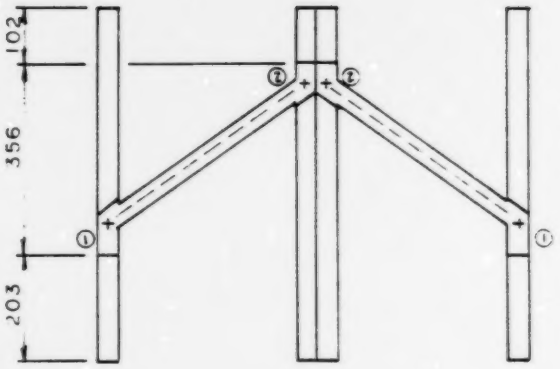


WEB DETAILS

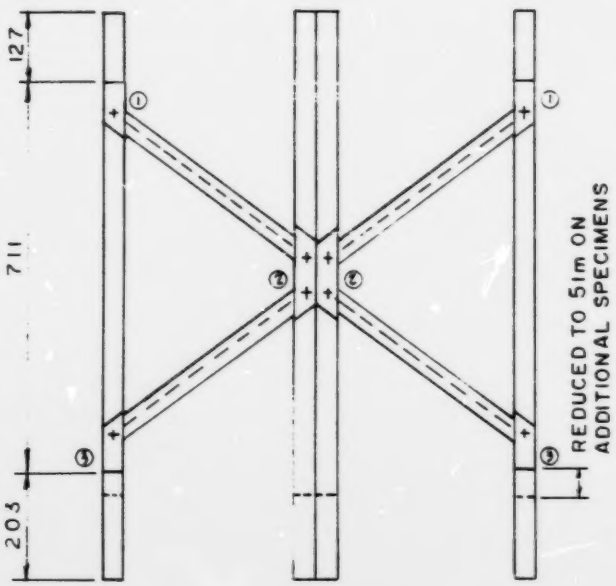
FIGURE 18- SHEAR PANEL SPECIMENS :
TRUSS TYPE MW 270mm DEPTH



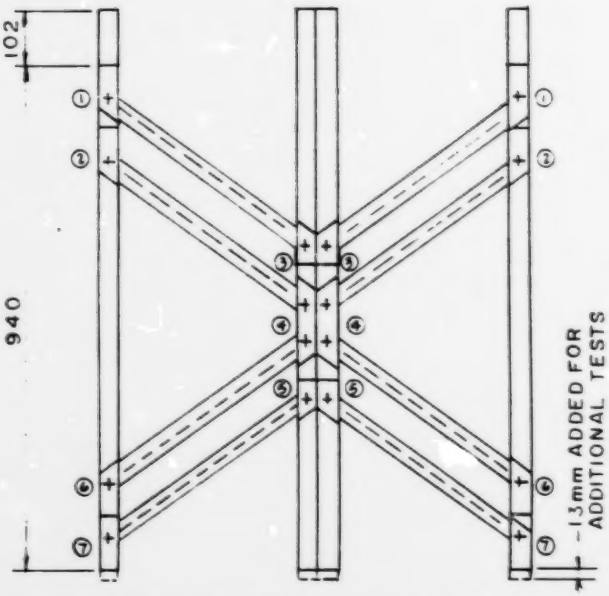
SPECIMEN E TENSION STRUT



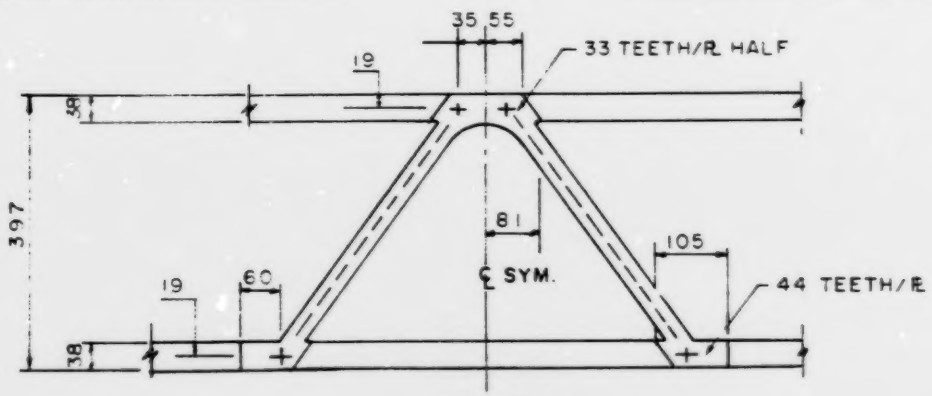
SPECIMEN F COMPRESSION STRUT



SPECIMEN G SIMPLE VEE

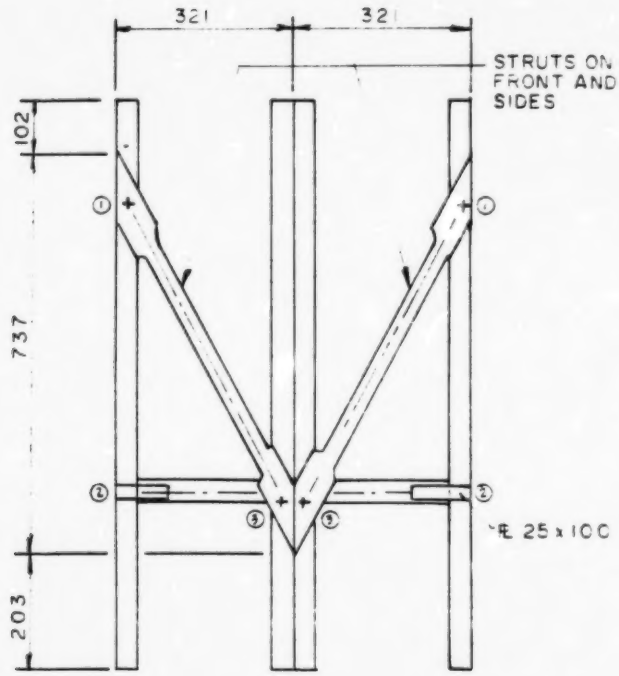


SPECIMEN H REINFORCED VEE

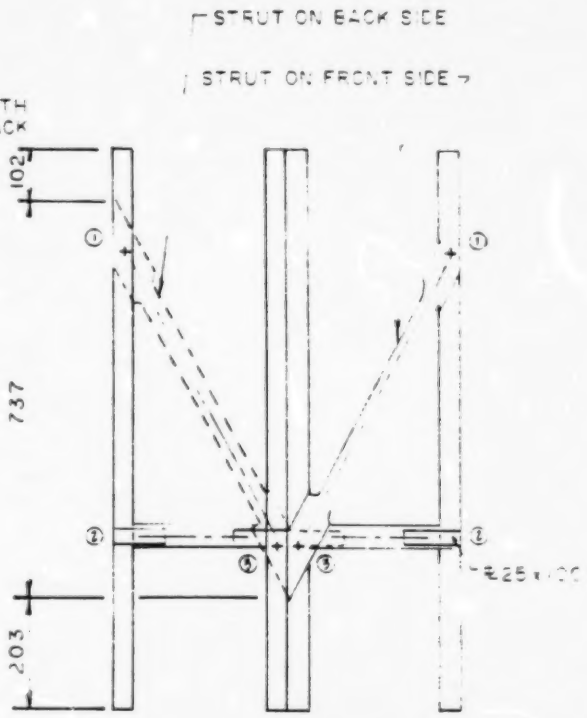


WEB DETAIL

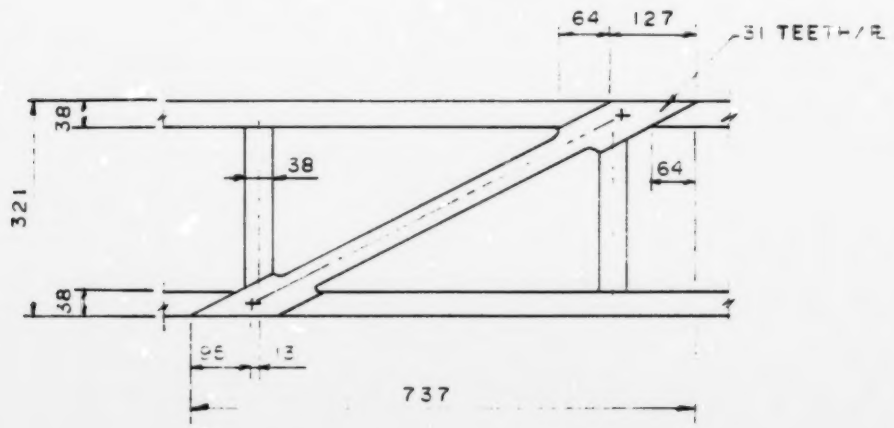
FIGURE 19 - SHEAR PANEL SPECIMENS : TRUSS TYPE MW 397mm DEPTH



SPECIMEN I DOUBLE TENSION STRUT

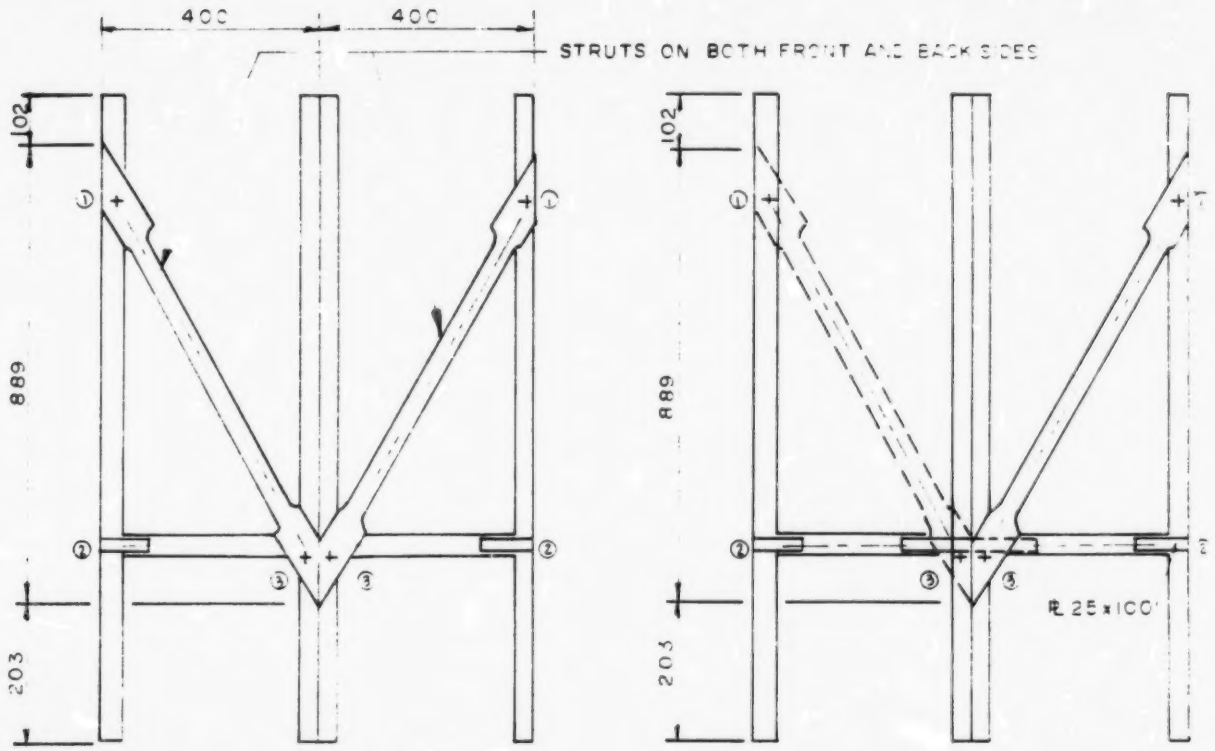


SPECIMEN J SINGLE TENSION STRUT



TENSION STRUT DETAIL

FIGURE 20 - SHEAR PANEL SPECIMENS
TRUSS TYPE TS 320mm DEPTH



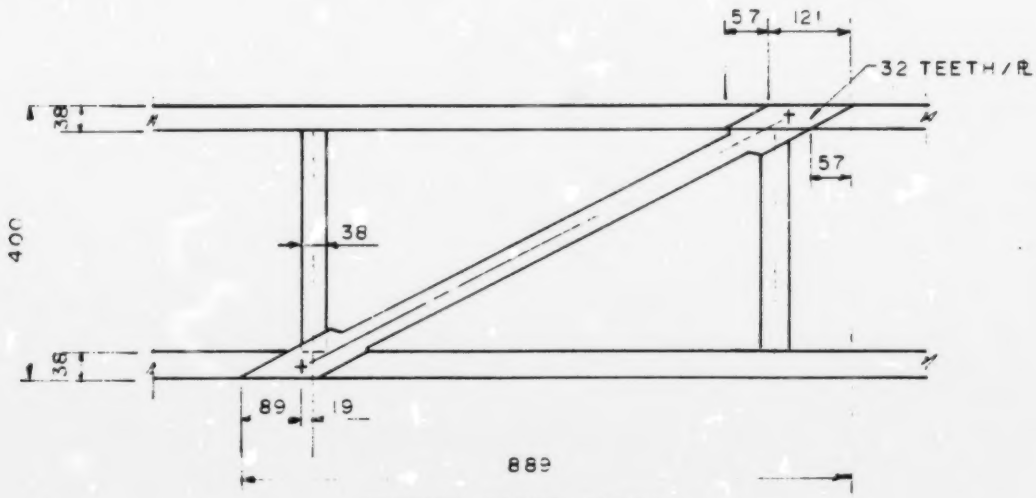
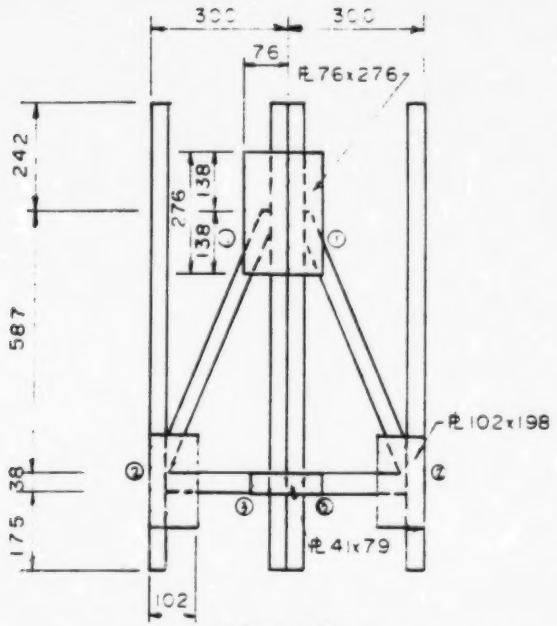
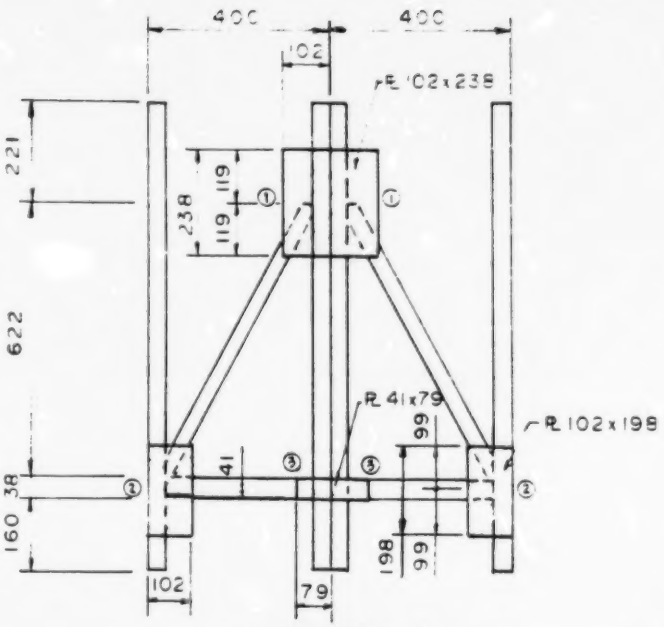


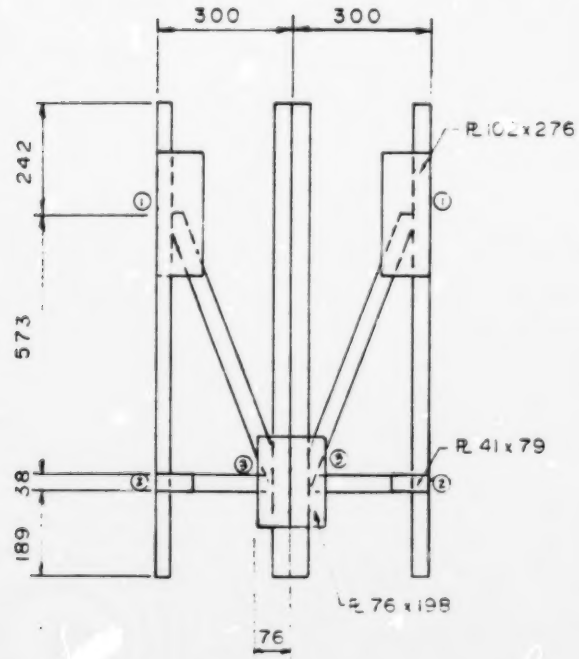
FIGURE 21 - SHEAR PANEL SPECIMENS
TRUSS TYPE TS, 400mm DEPTH



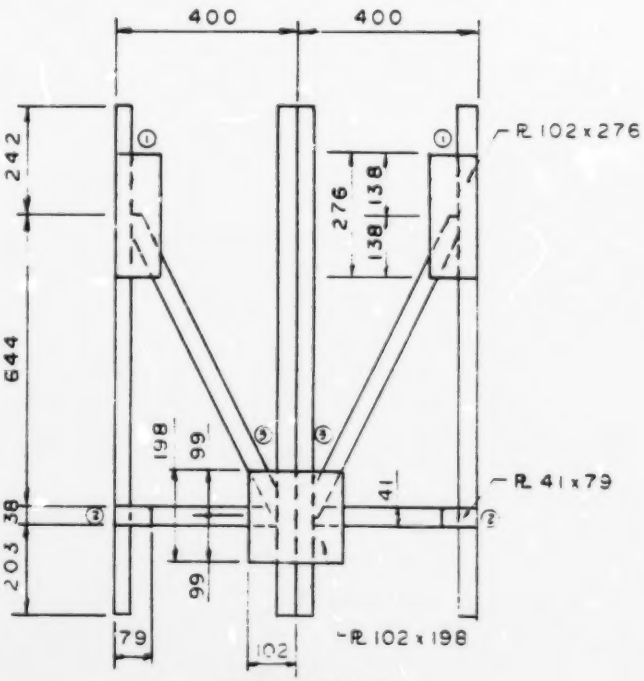
SPECIMEN M
300mm DEEP COMPRESSION WEB



SPECIMEN N
400mm DEEP COMPRESSION WEB



SPECIMEN O
300mm DEEP TENSION WEB



SPECIMEN P
400mm DEEP TENSION WEB

FIGURE 22 - SHEAR PANEL SPECIMENS TRUSS TYPE WW,
300 AND 400 mm DEPTHS

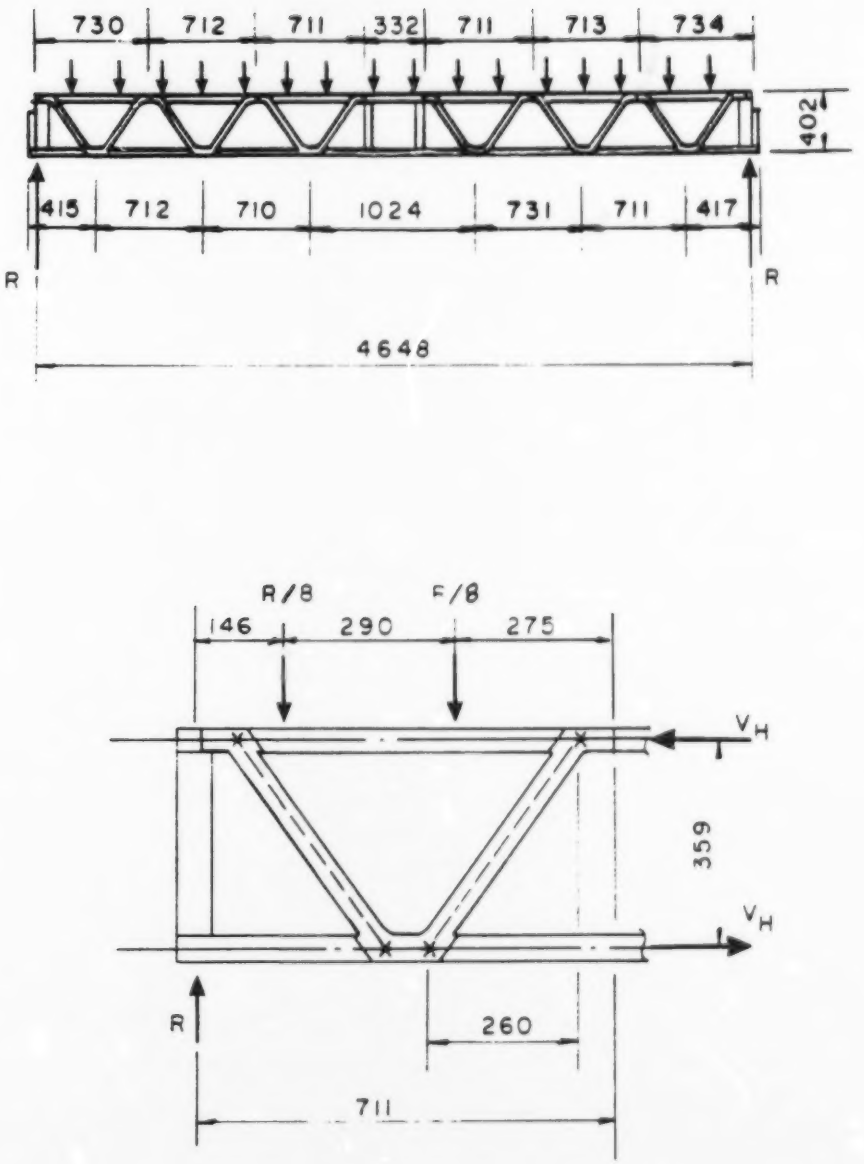
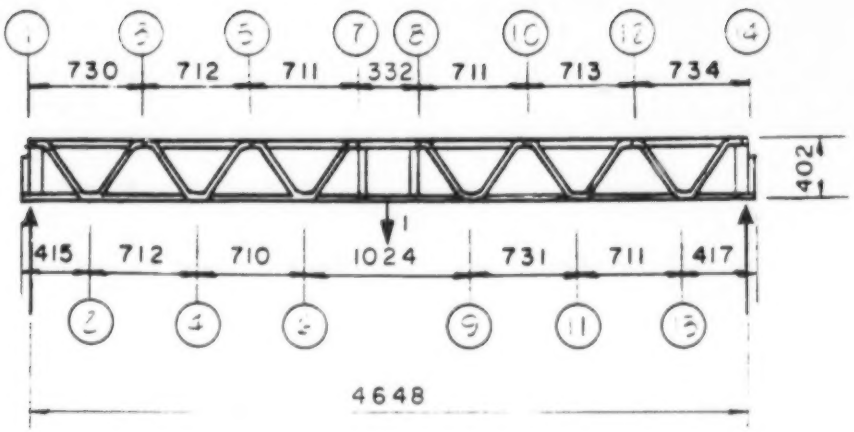


FIGURE 23 - END PANEL OF AN MWC TRUSS



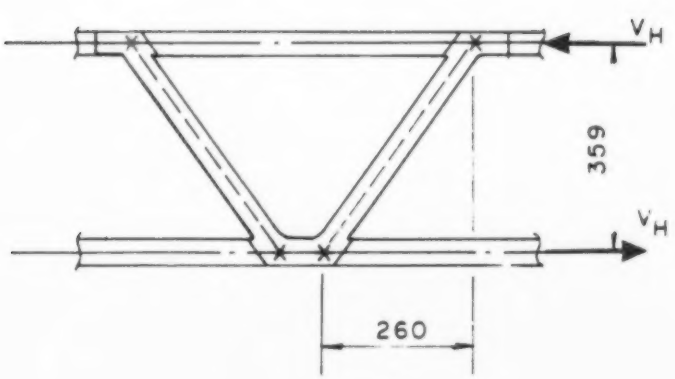


FIGURE 24 - MWC TRUSS AND TYPICAL WEB UNIT

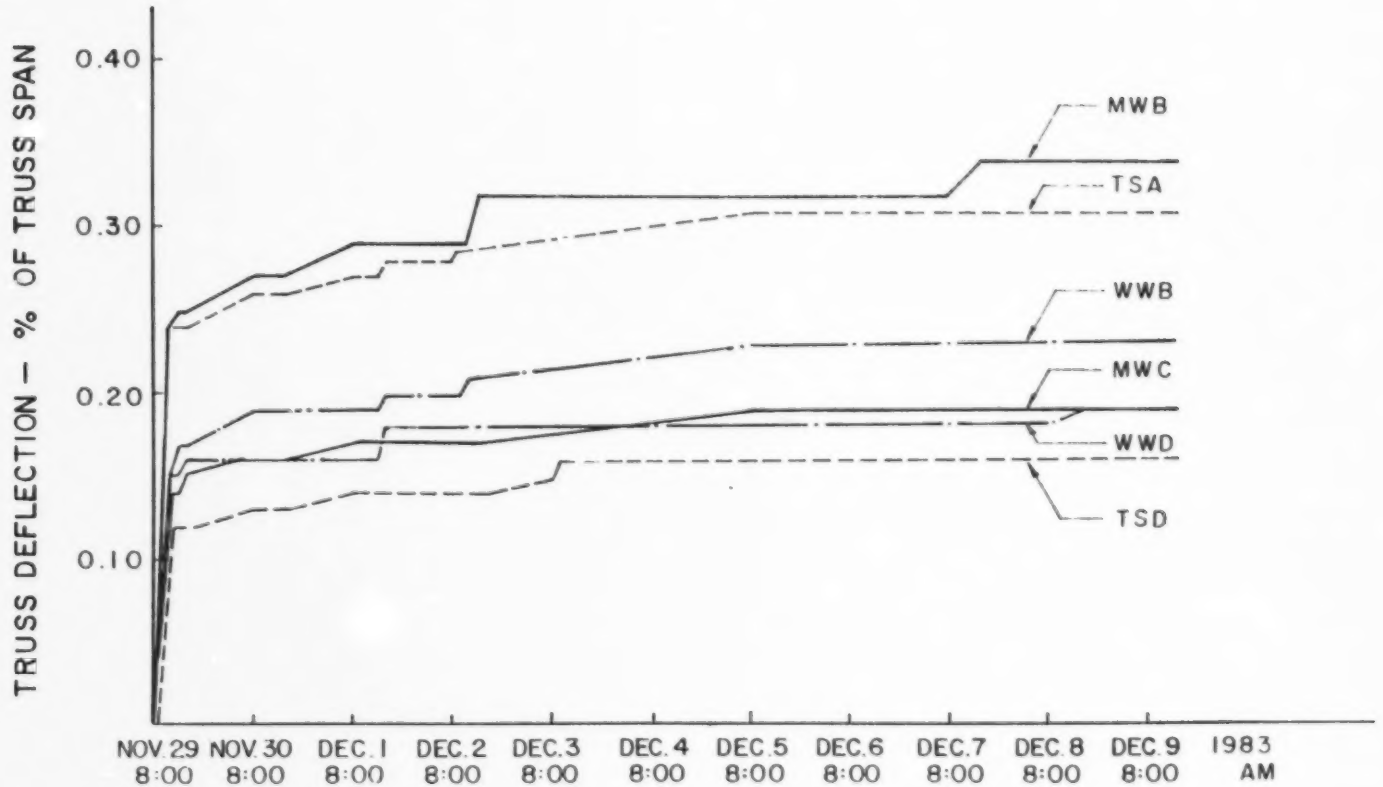


FIGURE 25 - TRUSS DEFLECTIONS DURING THE FIRST 11 DAYS OF LONG-TERM LOADING

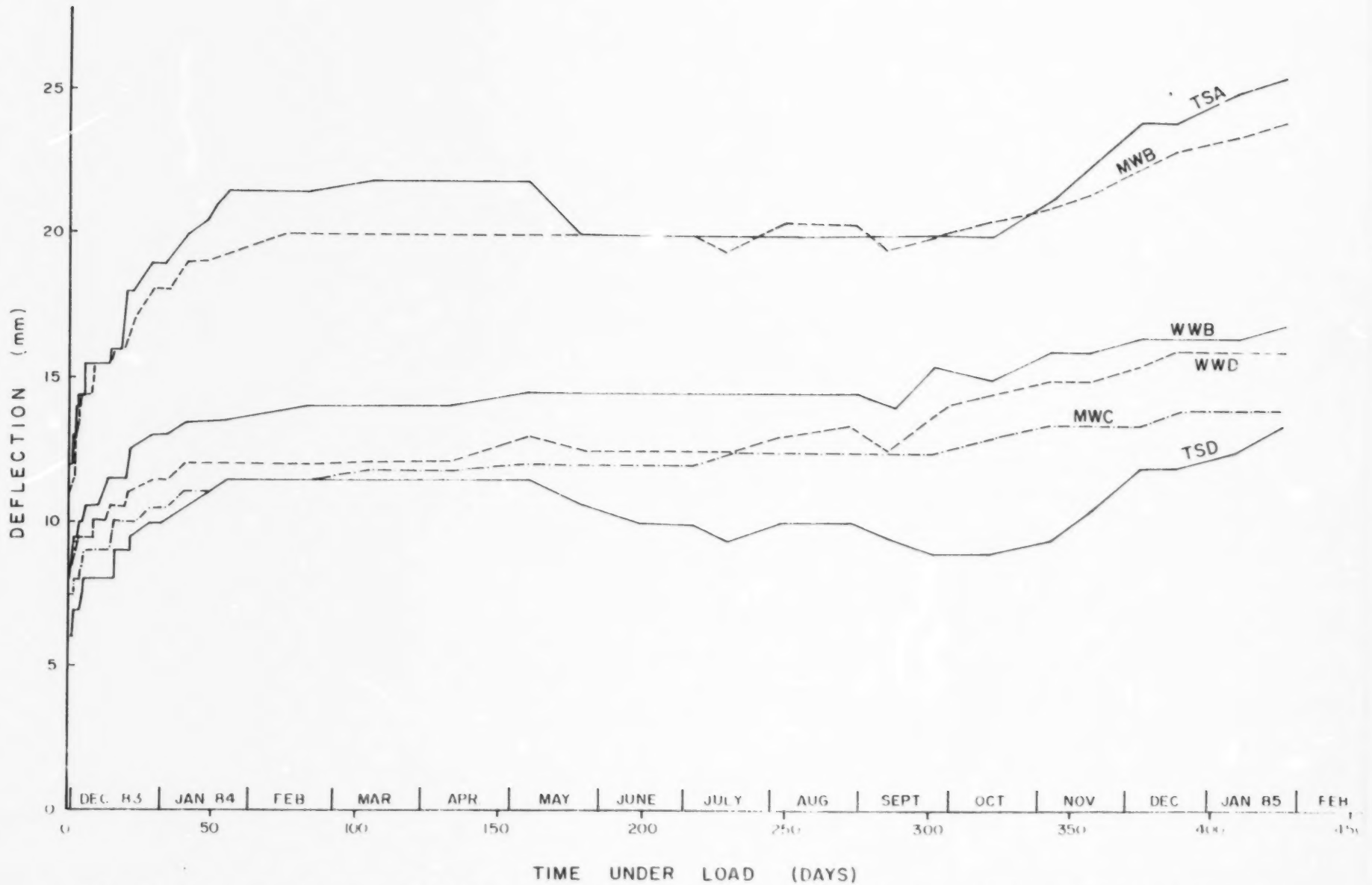


FIGURE 26 - TRUSS DEFLECTIONS DURING THE FIRST 450 DAYS OF LONG-TERM LOADING

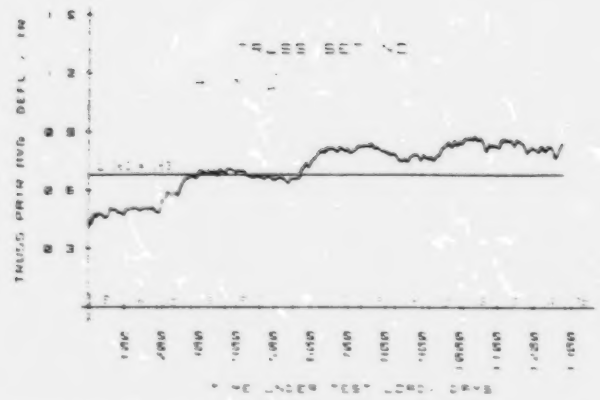


Figure 3.1. Truss type 1.

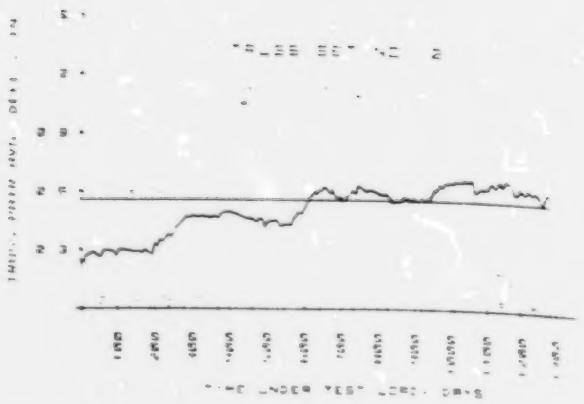


Figure 3.2. Truss type 2.

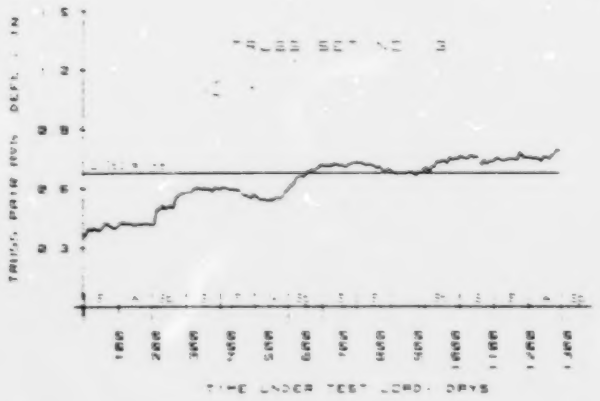


Figure 3.3. Truss type 3.

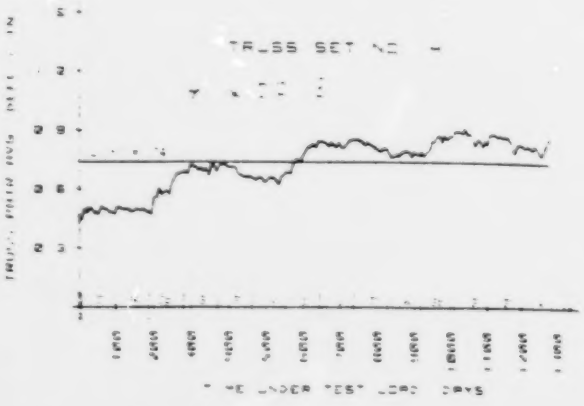


Figure 3.4. Truss type 4.

Long-term test deflection data for the period beginning September 15, 197[illegible]. The test load is 20 pounds per square foot, which is one-half live load. Clear span divided by 360 is shown as a horizontal reference line. While the basic time axis is in days, seasons of the year are also marked with longer vertical line segments.

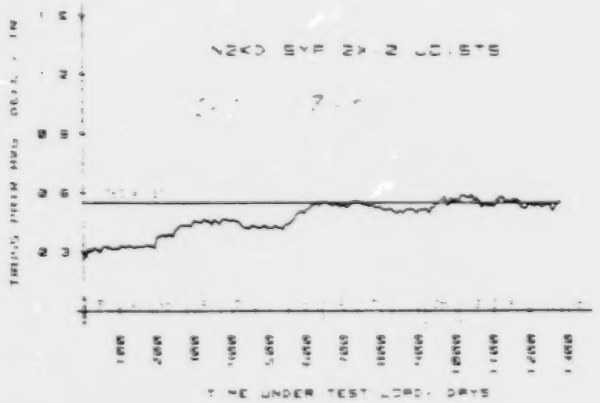


Figure 3.5. No. 2 KD SYP 2 x 10 joists.

FIGURE 27 - EXTRACT FROM THE REPORT BY SUDDARTH ET AL

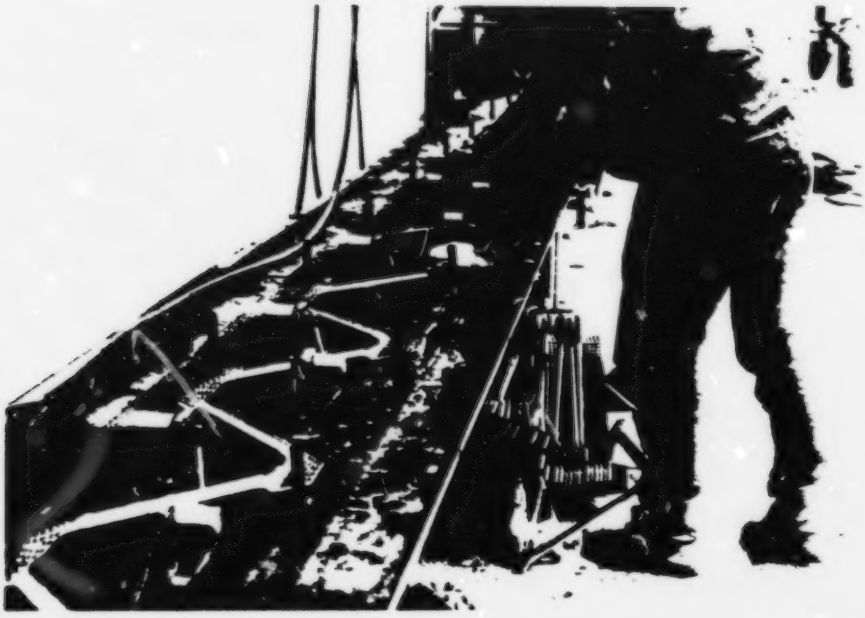


Photo 1
Fabrication of MW truss - one set of metal vee's.

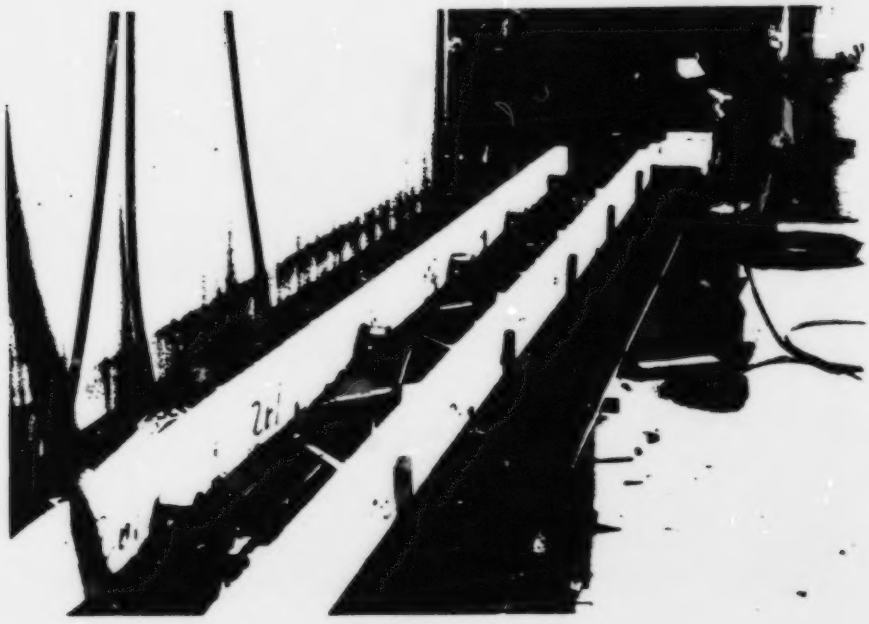


Photo 2
Fabrication of MW truss (cont'd) - lumber chords added.

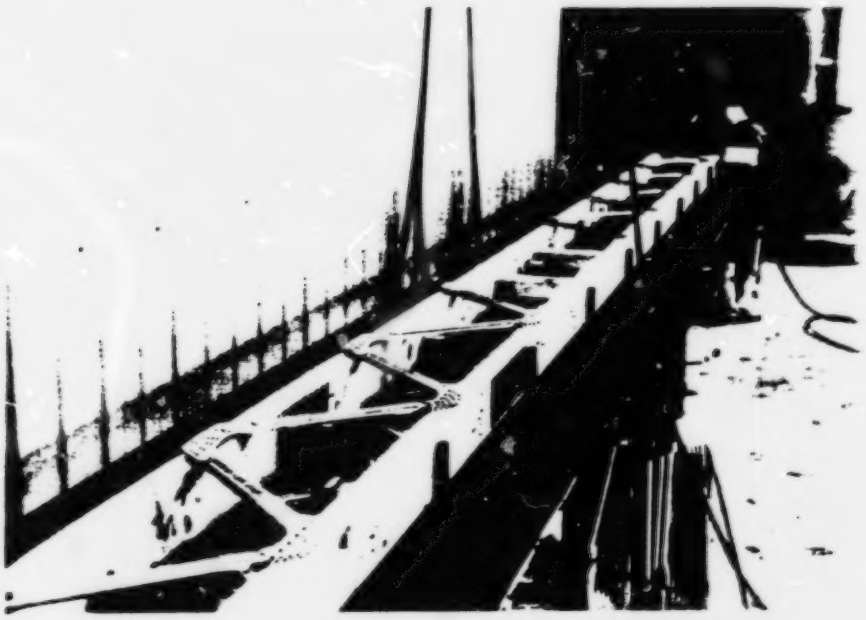


Photo 3
Fabrication of MW truss (cont'd) - second set of metal vee's added.

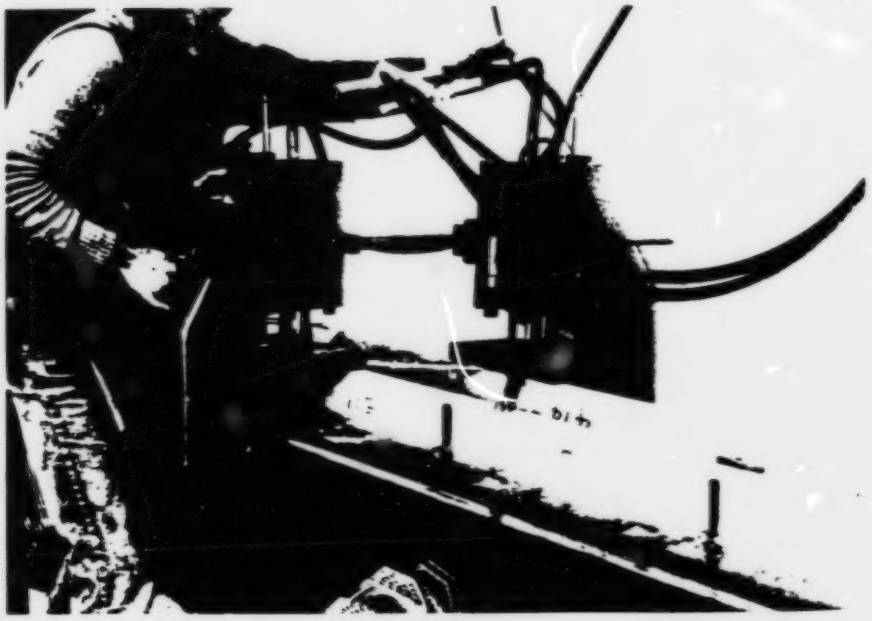


Photo 4
Fabrication of MW truss (cont'd) - pressing.

Photo 5
Fabrication of TS truss - one set of metal diagonals and lumber chords.

Photo 6
Fabrication of TS truss (cont'd) - lumber verticals added.

Photo 7
Fabrication of TS truss (cont'd) - second set of metal diagonals added.

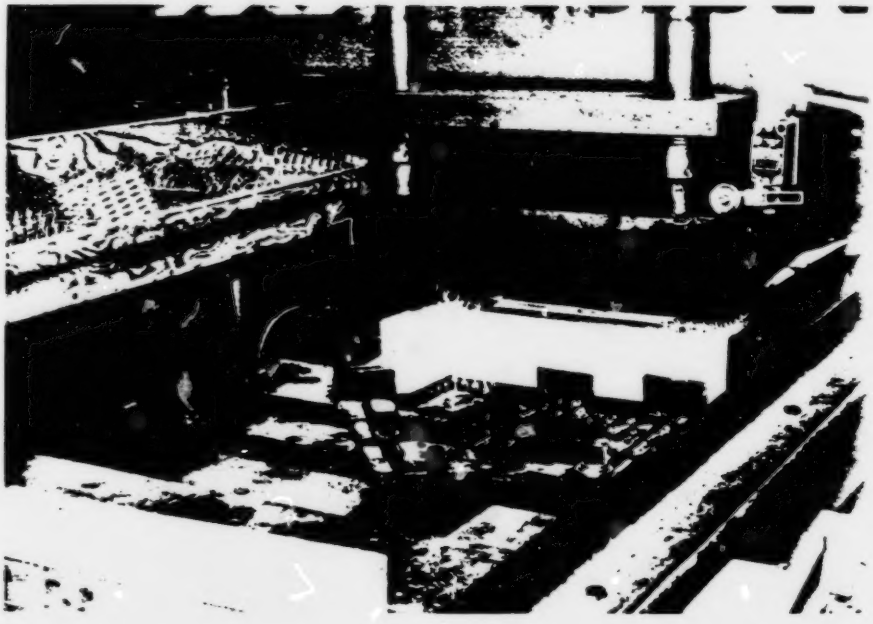


Photo 8
Fabrication of TS truss (cont'd) - pressing.

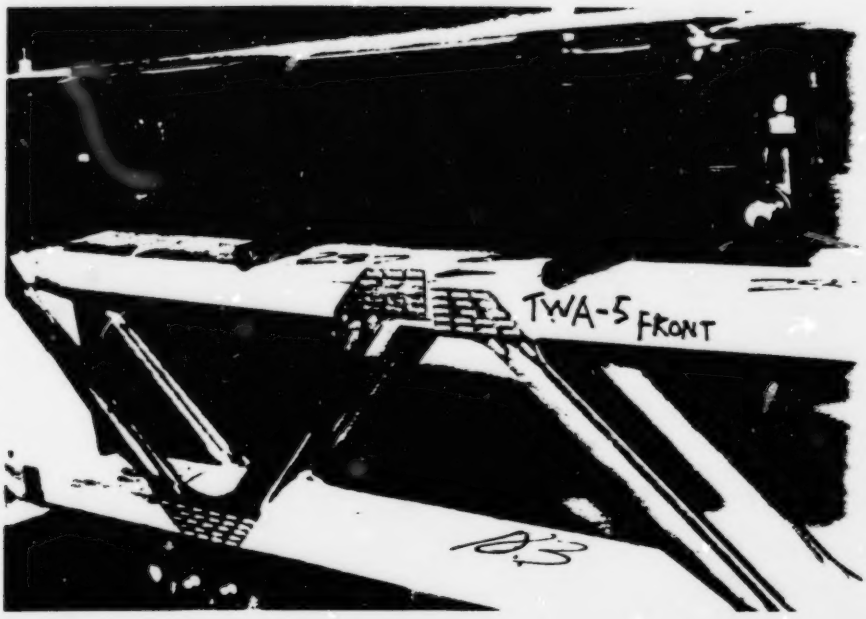


Photo 9
Truss MWA-5: failure mode is web tooth pullout in compression at top chord joint 3.

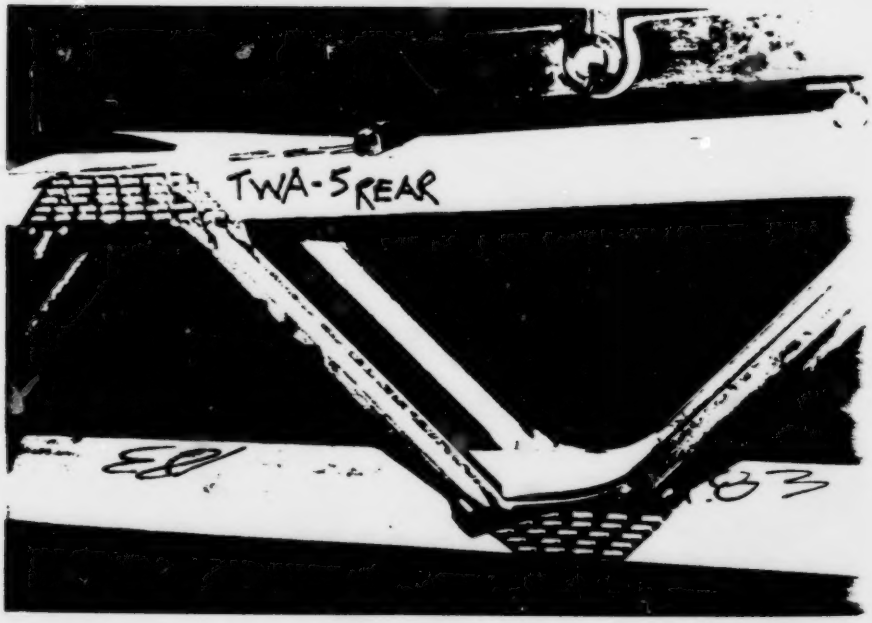


Photo 10
Truss MWA-5: secondary failure is buckling of the compression leg of the metal vee.

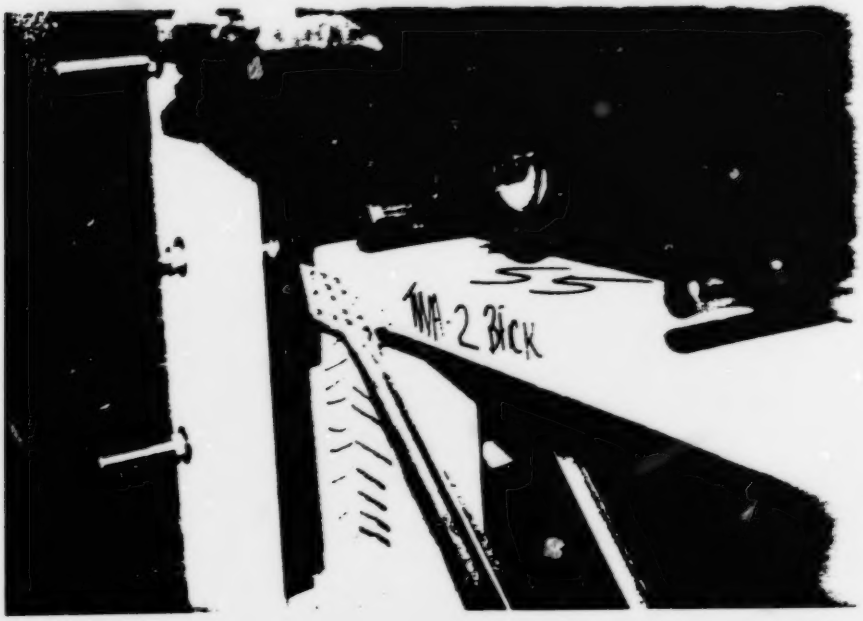


Photo 11
Truss MWA-2: failure mode is web tooth pullout in tension at joint 22.

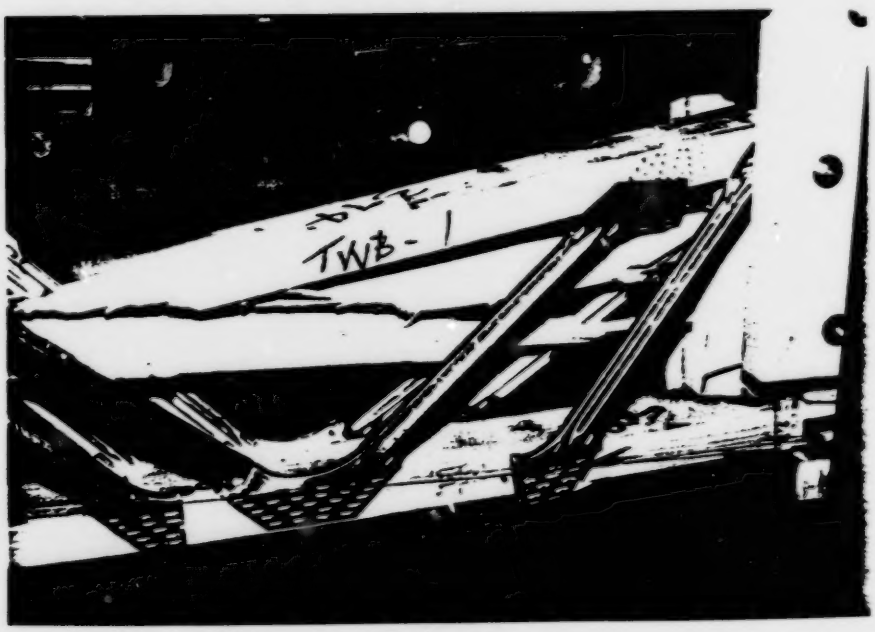


Photo 12
Truss MWB-1: the primary failure mode is web tooth pullout in tension at joint 14.

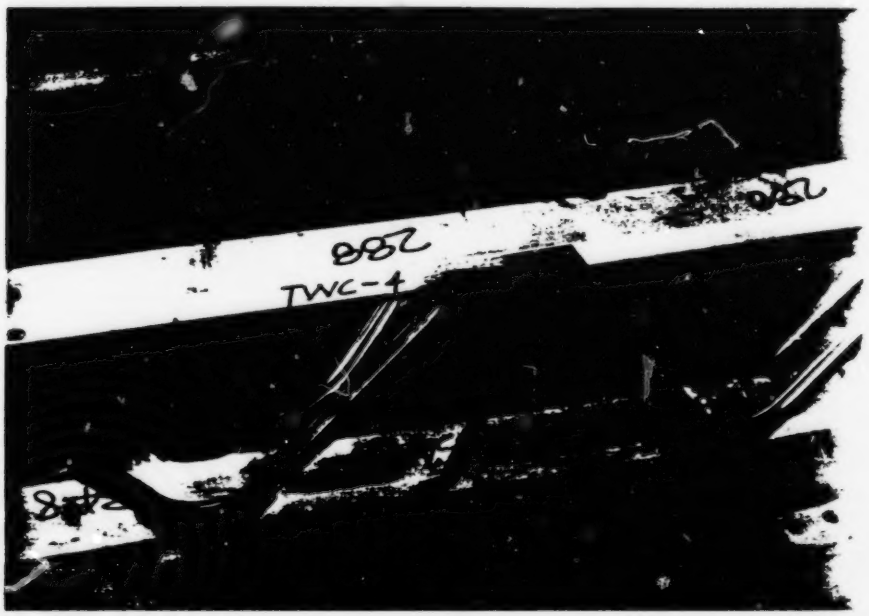


Photo 13
Truss MWC-4: failure mode is buckling of the first compression diagonal plus web tooth pullout in compression.

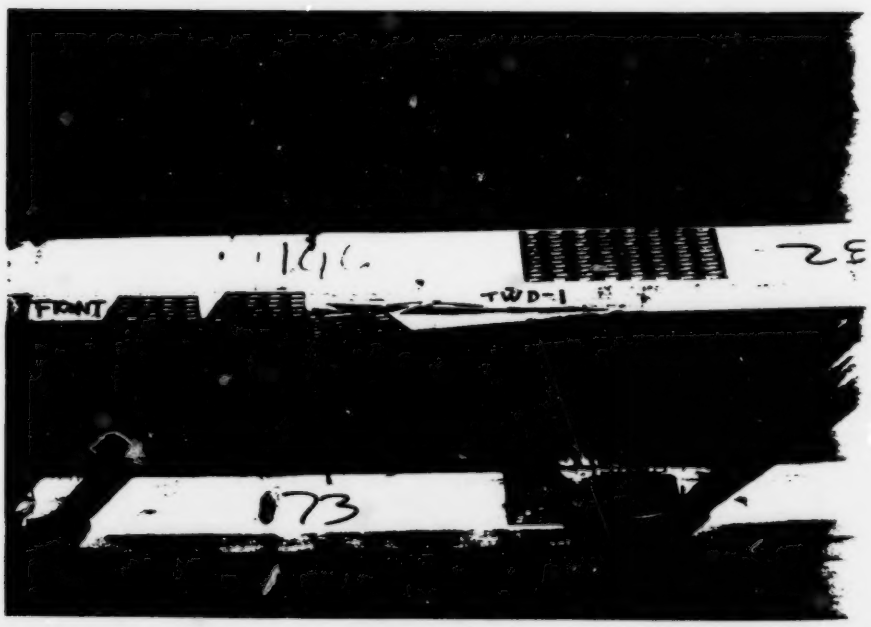


Photo 14
Truss MWD-1: failure mode is combined tooth pullout and wood pullout at the top of the second tension diagonal (joint 3).

Photo 15
Truss TSA-5: failure mode is tooth withdrawal at the bottom chord splice.

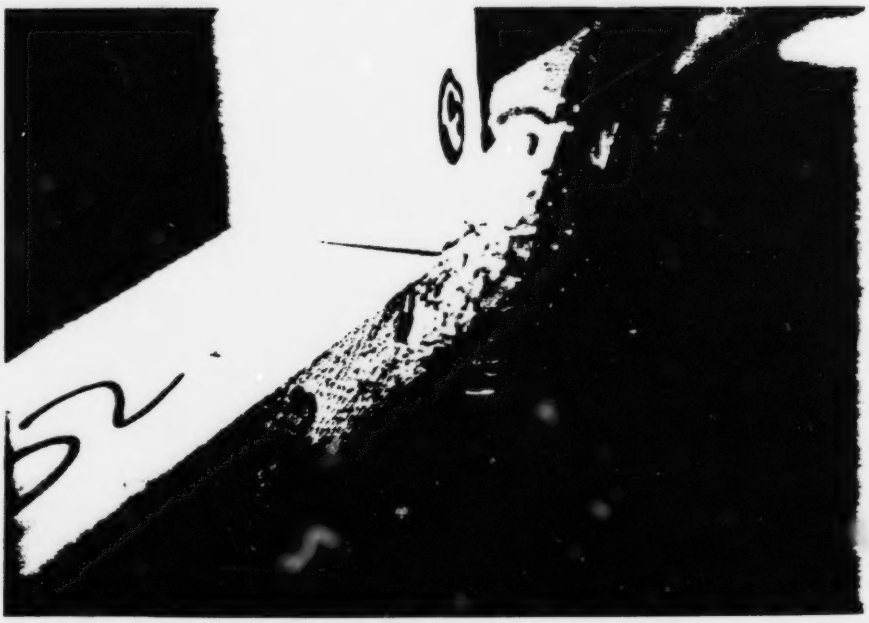


Photo 16
Truss TSB-3: tooth withdrawal from the bottom chord at the lower end of the first tension diagonal.

Photo 17
Truss TSB-5: metal tension failure in the grip area of the first tension diagonal.

Photo 18
Truss TSD-1: truss plate tooth withdrawal from the bottom chord at the lower end of the first tension diagonal.

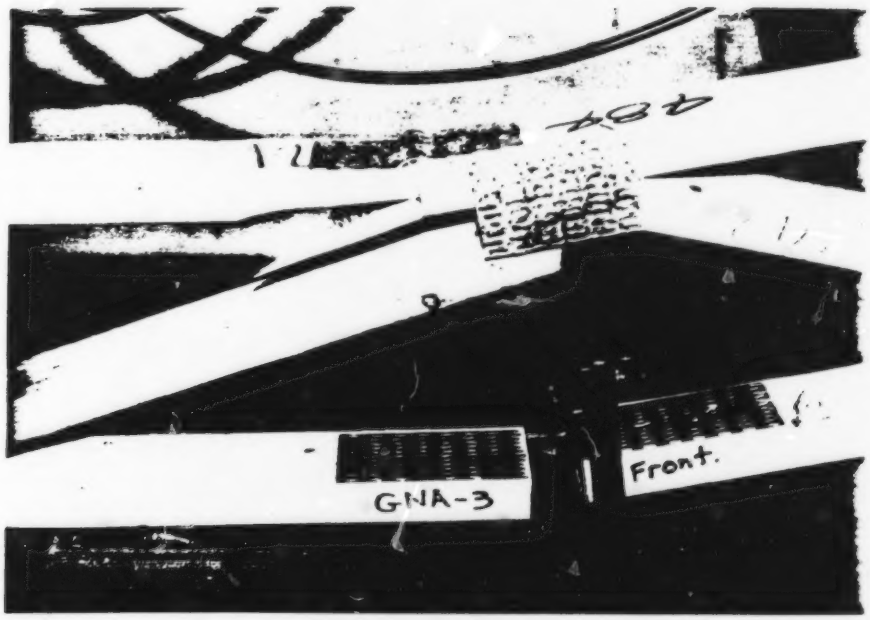


Photo 19
Truss WWA-3: metal tension failure in the bottom chord splice.

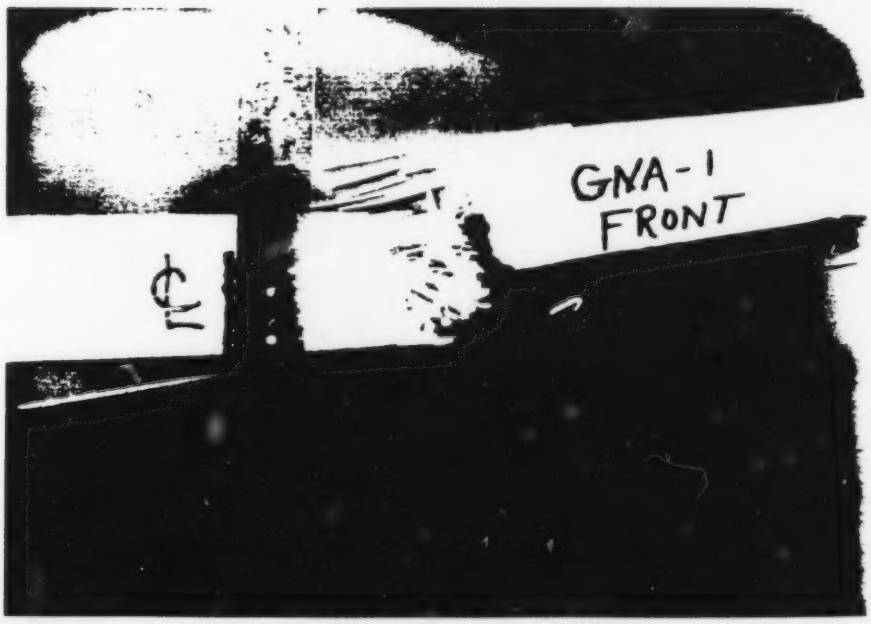


Photo 20
Truss WWA-1: tension failure in the bottom chord at a large knot.

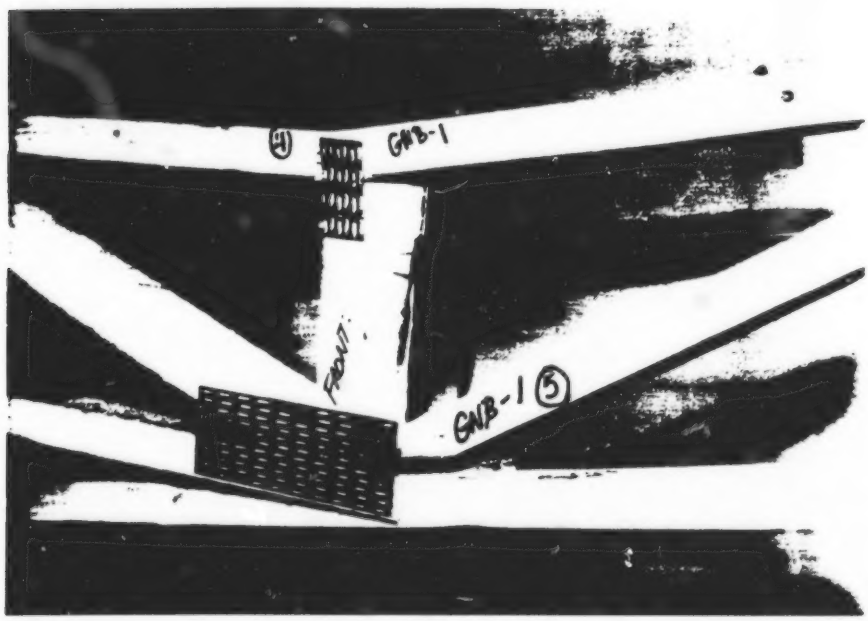


Photo 21
Truss WWB-1: wood failure in the bottom chord at joint 5.

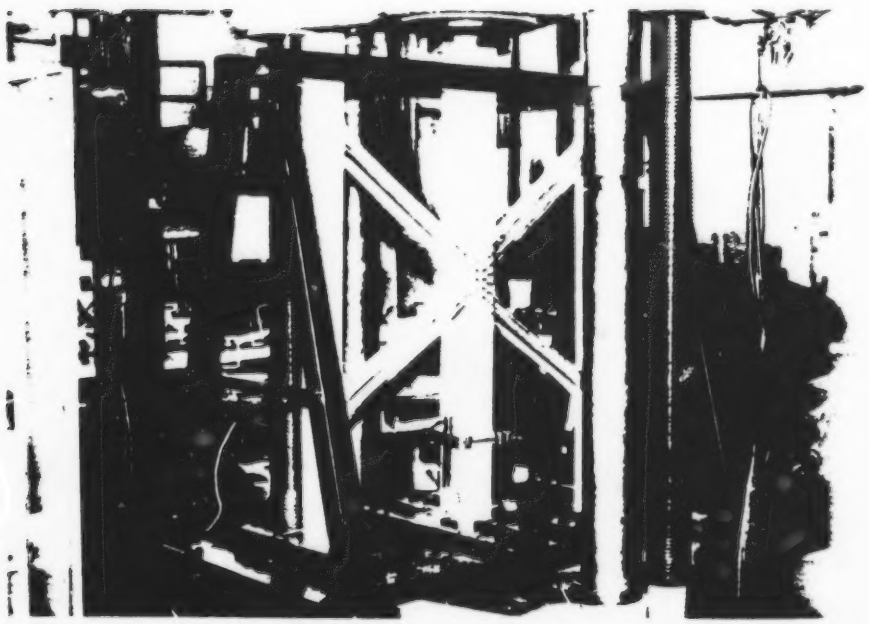


Photo 22
Shear panel test set-up.

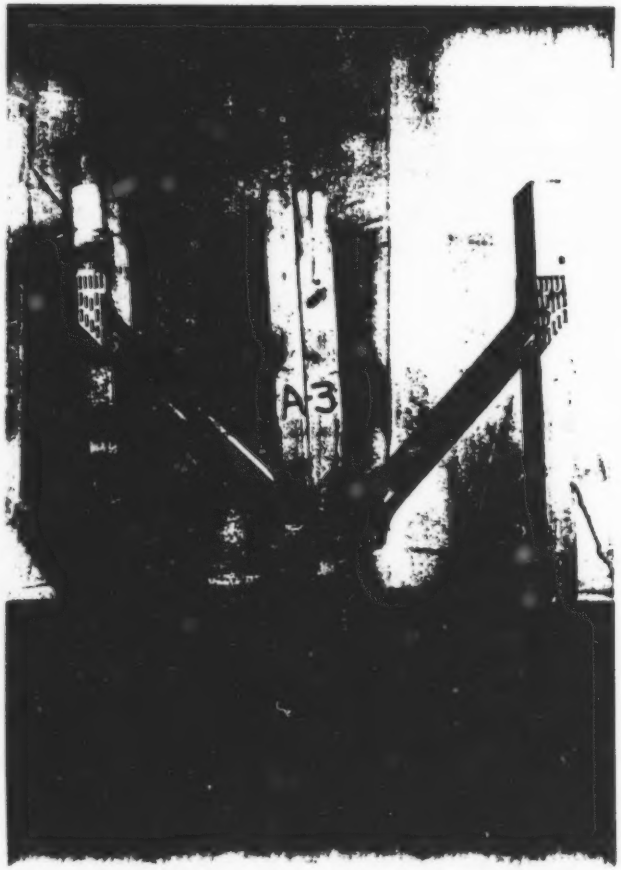


Photo 23
Shear panel A-3: wood splitting under the plate at joint 2 and tooth withdrawal.

Photo 24
Shear panel B-1: tooth withdrawal and web buckling.

Photo 25
Shear panel C-20: tooth withdrawal at joint 3 plus web buckling.

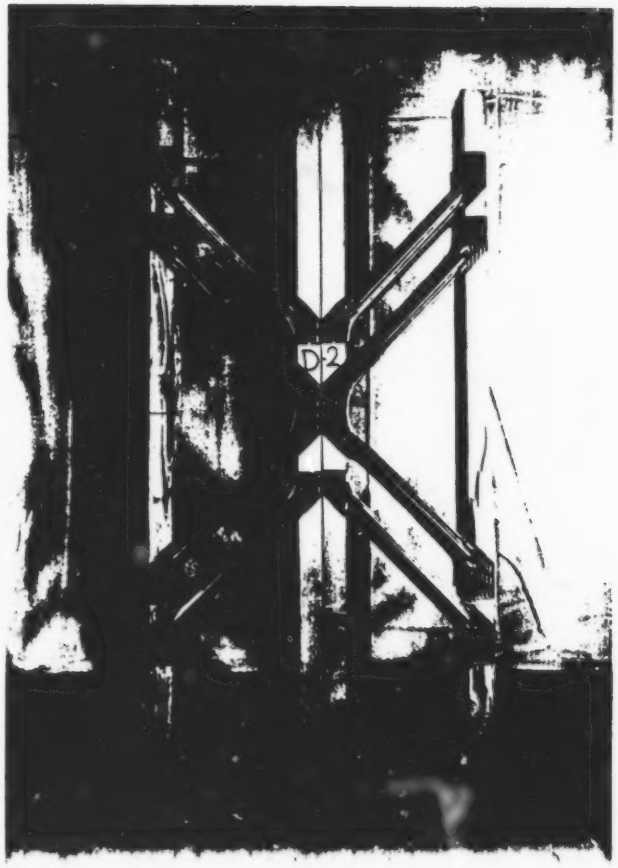


Photo 26
Shear panel D-2: splitting of the wood under the plate at joints 6 and 7.

Photo 27
Shear panel E-3: tooth withdrawal with wood at joint 2.

Photo 28
Shear panel F-2: tooth withdrawal at joint 2 plus web buckling.

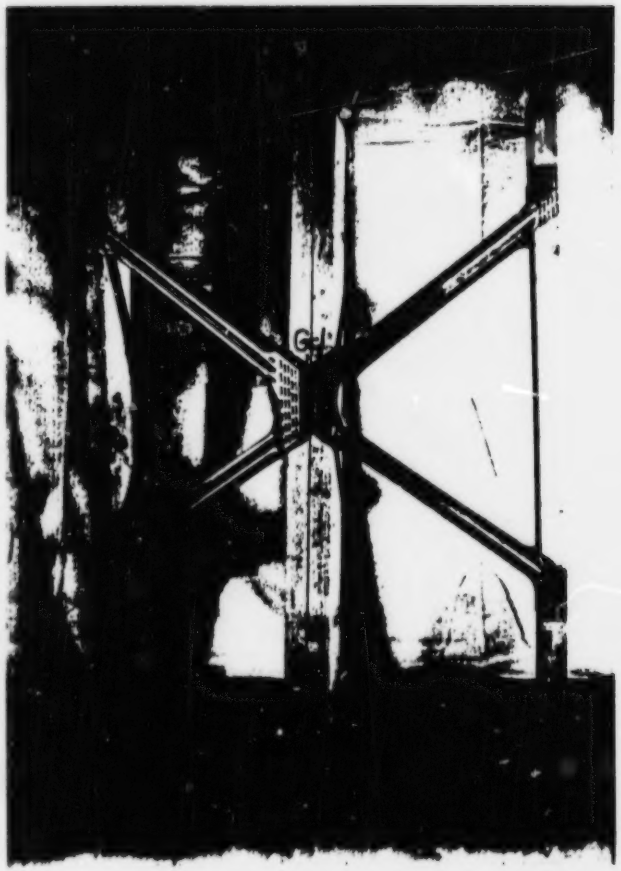


Photo 29
Shear panel G-1: tooth withdrawal at joint 2 plus web buckling.

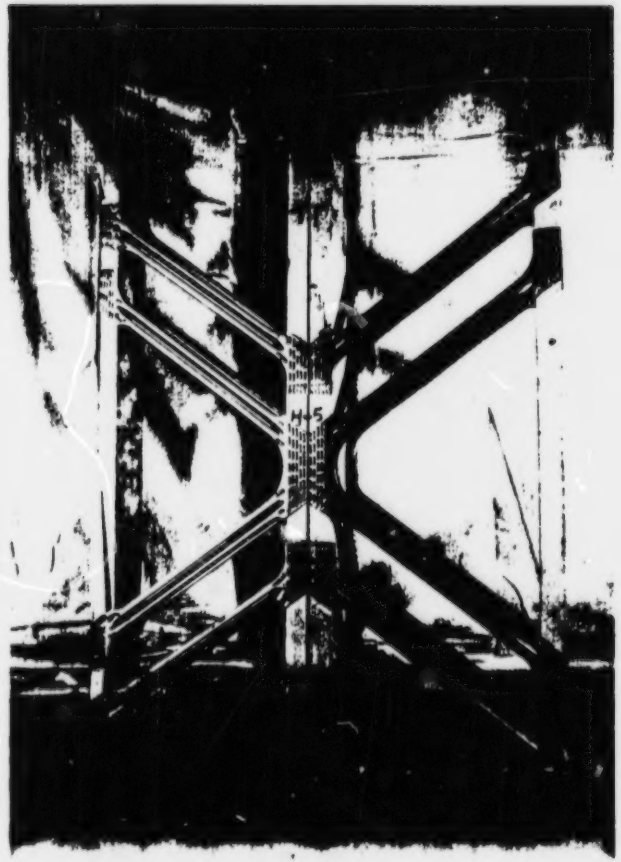


Photo 30
Shear panel H-5: wood split under plate at joints 5 and 6.

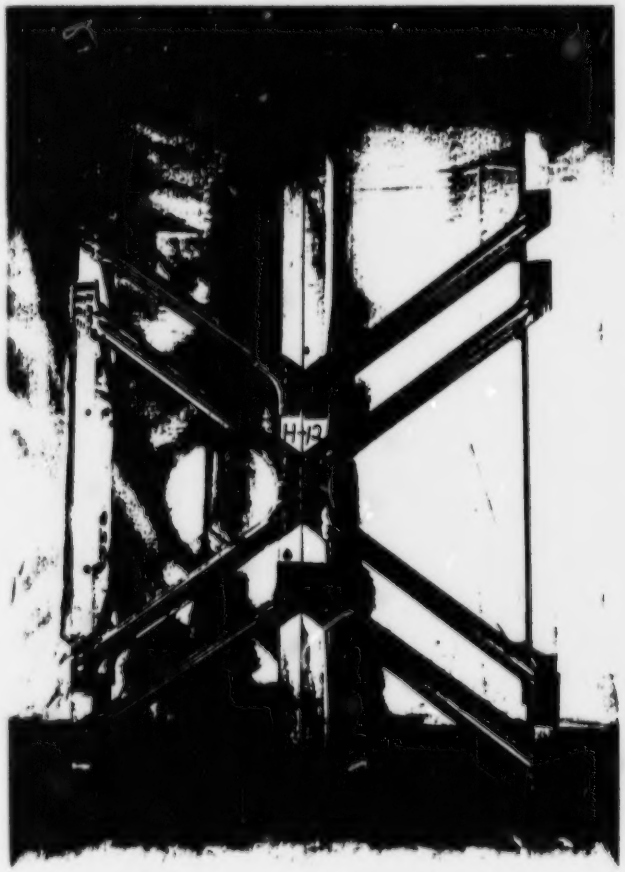


Photo 31
Shear panel H-12: tooth withdrawal at joint 7 plus web buckling.

Photo 32
Shear panel I-14: tooth withdrawal with wood at joint 1.

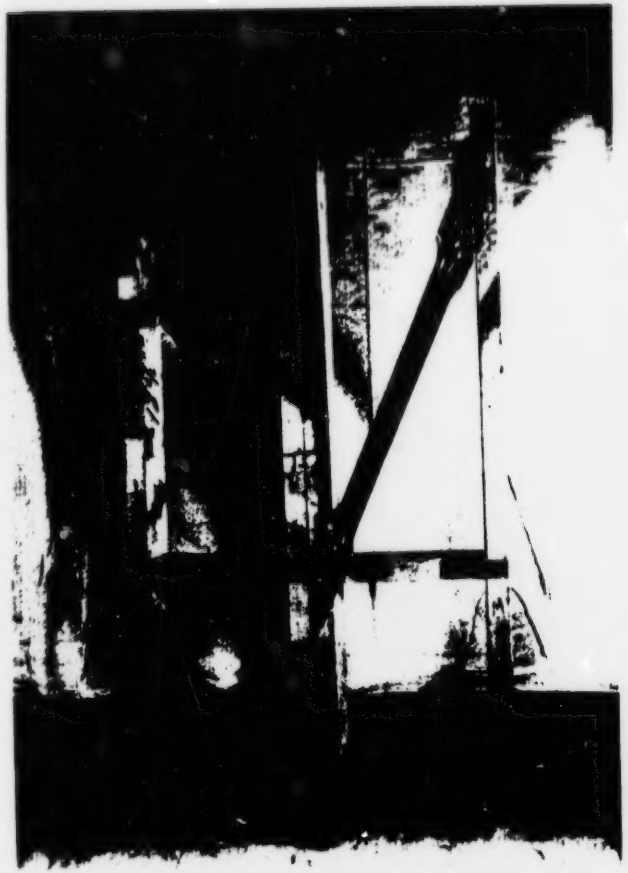


Photo 33
Shear panel J-7: tooth withdrawal at joint 1.

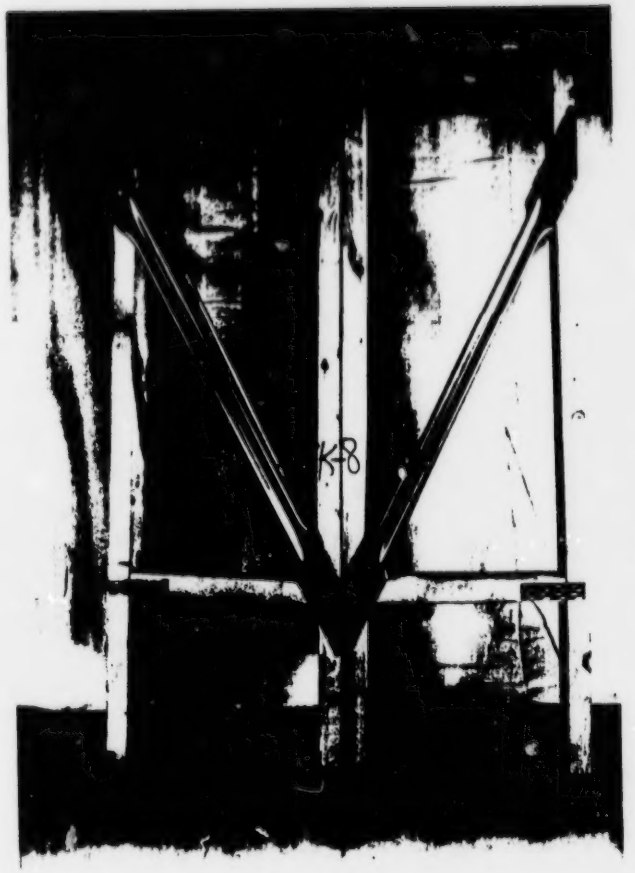


Photo 34
Shear panel K-8: tooth withdrawal at joint 1.

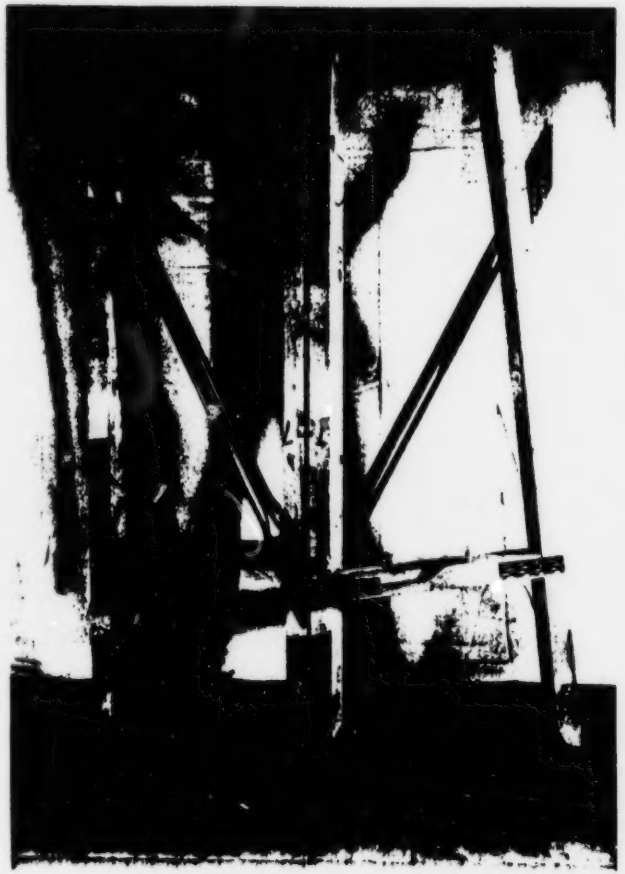


Photo 35
Shear panel L-1: tooth withdrawal at joints 1 and 3.

Photo 36
Shear panel M-1: plate buckling at joint 2.

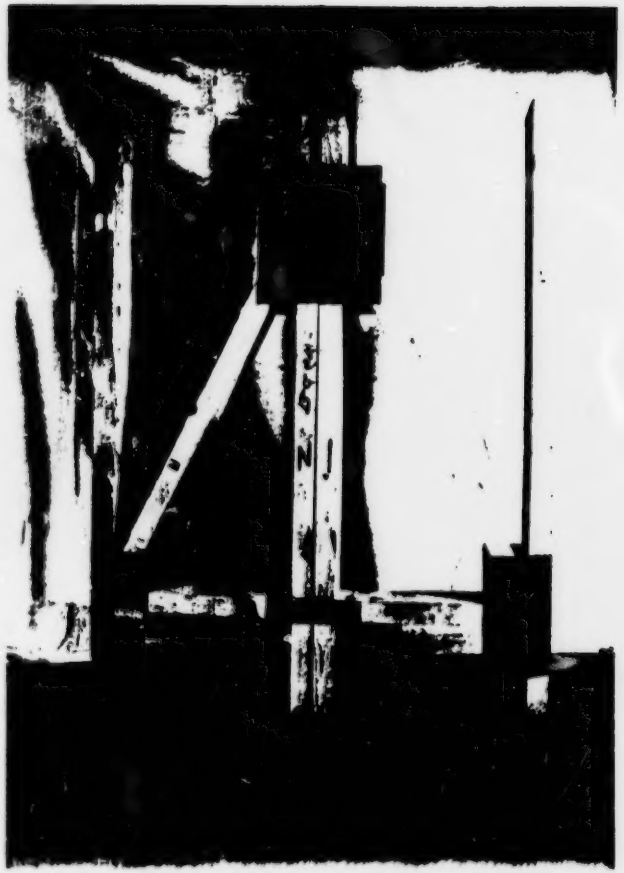


Photo 37
Shear panel N-1: plate buckling at joints 1 and 2.

Photo 38
Shear panel 0-4: tooth withdrawal at joint 3.

Photo 39
Trusses prepared for long-term tests.

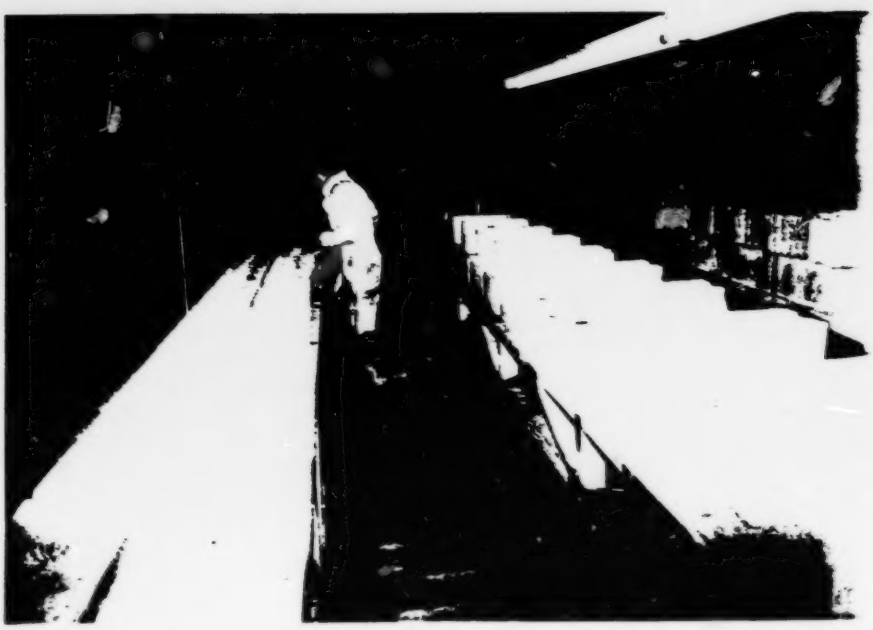


Photo 40
Masonry blocks used for long-term loads.